Berryessa Creek Watershed Hydrology Report

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Prepared for:

Coyote Watershed Program
Santa Clara Valley Water District

and

US Army Corps of Engineers
Sacramento District

Berryessa Creek Watershed
Hydrology Report
Addendum 1

Lower Berryessa Creek Project
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Addendum to the April 2003 Berryessa Creek Watershed Hydrology Report

Purpose

northwest hydraulic consultants, inc. (**nhc**) was retained by Winzler and Kelly (W&K) on behalf of the Santa Clara Valley Water District (District) to revise an existing HEC-HMS hydrologic model of the Berryessa Creek watershed to account for possible future drainage improvements along tributaries to Berryessa Creek.

Background

nhc developed an HEC-HMS hydrologic model of the Berryessa Creek watershed for the District in April 2003. That model assumed break out flow could occur along Piedmont, Los Coches, or Tularcitos Creeks if flows exceeded the existing channel capacity in those tributaries. Subsequent to the 2003 study, the District determined that hydrology for the Berryessa Creek basin should consider a condition without break out flows. For this reason the District directed W&K to retain **nhc** to modify the 2003 HEC-HMS future land use (full build out) conditions model to eliminate break out flow conditions previously modeled and reported in April 2003. This addendum summarizes the changes made to the original April 2003 HEC-HMS model and the results from the modified October 2006 HEC-HMS model of the Berryessa Creek Watershed.

Approach

In order to remove break out flow from the original 2003 HEC-HMS model the following steps were taken:

1. The April 2003 HEC-HMS future land use conditions (i.e. full build out assumption) model was retrieved from the project archives.
2. The HEC-HMS model was translated from HEC-HMS version 2.2.1 to version 3.0.1.
3. Results from the version 3.0.1 model were compared to results from the original version 2.2.1 model to ensure that the results reported in 2003 could be replicated within an acceptable tolerance using the most recent version of HEC-HMS.
4. The HEC-HMS model was updated to remove the breakout flows previously assumed in 2003.
5. The no breakout flow model was run to provide peak flows at the locations listed in Table 15 found in the April 2003 report as well as the additional locations listed in Winzler & Kelly's October 2005 technical memorandum to the District.

These steps are described in greater detail below.

The original model results were developed using HEC-HMS version 2.2.1. Because version 2.2.1 has been superseded and is no longer being actively supported by HEC, **nhc** converted the model into the current HEC-HMS code (version 3.0.1). To ensure that the conversion did not cause any changes in flows, results from the new version 3.0.1 model were compared with the results from the original 2.2.1 version. Comparison of the version 3.0.1 results confirmed that flows at all points matched the results from the April 2003 study to within 0.1 cfs. Once it was known that the HEC-HMS version 3.0.1 model could reproduce the earlier results, the model input records were modified to remove the breakout flow assumptions previously specified on Piedmont, Los Coches, and Tularcitos tributaries as per the request from the District and W&K.

Break out flows in the original HEC-HMS model were simulated using diversion elements on Piedmont, Los Coches, and Tularcitos Creeks. The reaches representing these creeks used the Modified Puls routing method, which requires a storage-discharge table for each routing reach. Therefore, when the flow diversions were removed in the modified HEC-HMS model, new storage-discharge tables were required for these creeks. For the present study the required storage-discharge tables were developed using available hydraulic information and the following HEC-RAS models:

- Berryessa Creek Alternative 2 model provided by W&K,
- Tularcitos Creek Alternative 2 model provided by W&K,
- **nhc** model of Piedmont Creek developed for the 2003 study, and
- **nhc** model of Los Coches Creek developed for the 2003 study.

W&K's Berryessa Creek Alternative 2 HEC-RAS model was extended upstream by **nhc** to the confluence of Piedmont Creek using seven cross sections from the HEC-2 model of Berryessa Creek provided to **nhc** by the District in 2003, with appropriate elevation adjustments.

New storage-discharge tables were developed using an iterative process to balance flows between the modified Berryessa watershed HEC-HMS model and HEC-RAS models listed above. Flows from the HEC-HMS model were inserted into the HEC-RAS model of Berryessa Creek and used to establish tailwater ratings for the tributaries. These tailwater elevations were then used in the tributary models to calculate new storage-discharge tables for Tularcitos, Los Coches, and Piedmont Creeks. These updated storage-discharge tables were then inserted into the HEC-HMS model and run to produce new flow estimates for the tributaries and mainstem of Berryessa Creek. The new flows were then reinserted into the HEC-RAS models and the process was repeated until the flows calculated by the HEC-HMS model converged. The new storage-discharge tables are shown in Table 13a on the following page.

New flows were computed with the newly updated future land use (full build out) condition HEC-HMS model with the assumption that no breakout flows would occur on Piedmont, Los Coches, or Tularcitos Creeks. Results from these simulations for the 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500- year recurrence intervals are summarized below in revised Table 15a.

Table 13 a. Modified Puls Parameters
Updated Storage-Discharge Tables for Piedmont, Los Coches, and Tularcitos Creeks

RP4-B17 Piedmont Creek

Storage (ac-ft)	**Discharge** (cfs)
0	0
1.20	41
7.18	301
10.08	470
11.30	539
15.09	842
17.63	1084
21.70	1330
25.70	1671
29.18	1880

RT4-T3 Tularcitos Creek

Storage (ac-ft)	**Discharge** (cfs)
0	0
2.20	27
8.78	230
13.18	364
15.77	448
23.13	664
29.08	865
40.63	1203
54.47	1332
70.52	1565

RLC-B17a Los Coches Creek

Storage (ac-ft)	**Discharge** (cfs)
0	0
0.19	36
0.87	252
1.33	411
1.65	541
2.51	781
3.33	996
4.87	1275
6.5	1597
7.63	1865

Revised Table 15a. 24-Hour Storm Flood Frequency Quantiles

Location	Drainage Area (sq mi)	HMS Node	Recurrence Interval (yrs)	Future Conditions (cfs)[1]
Lower Penitencia Creek Above Confluence with Coyote Creek	29.1	B33	2	2,480
			5	3,640
			10	4,310
			25	5,900
			50	6,980
			100	8,720
			200	10,790
			500	12,080
Lower Penitencia Creek Below Confluence with Berryessa Creek	28.2	B29	2	2,320
			5	3,490
			10	4,160
			25	5,760
			50	6,840
			100	8,400
			200	10,770
			500	12,080
Berryessa Creek at Downstream End Immediately Above Confluence w/ Lower Penitencia Creek	22.4	B27	2	1,720
			5	2,680
			10	3,160
			25	4,500
			50	5,490
			100	6,960
			200	9,040
			500	10,210
Berryessa Creek at North Abel Street Below Confluence with Wrigley-Ford Ditch	19.4	B25	2	1,590
			5	2,390
			10	2,810
			25	3,970
			50	4,820
			100	6,260
			200	7,720
			500	9,270
Berryessa Creek Berryessa Pump Station	17.8	B23	2	1,370
			5	2,120
			10	2,540
			25	3,590
			50	4,480
			100	5,710
			200	7,540
			500	8,670
Berryessa Creek Below Confluence with Tularcitos Creek	17.3	B21	2	1,280
			5	2,010
			10	2,420
			25	3,500
			50	4,360
			100	5,700
			200	7,140
			500	8,380

Revised Table 15a. 24-Hour Storm Flood Frequency Quantiles (Cont.)

Location	Drainage Area (sq mi)	HMS Node	Recurrence Interval (yrs)	Future Conditions (cfs)[1]
Berryessa Creek at Calaveras Boulevard	15.10	B19	2	1,090
			5	1,720
			10	2,060
			25	3,060
			50	3,830
			100	4,990
			200	6,090
			500	7,150
Berryessa Creek Below Confluence with Piedmont Creek	11.2	B17	2	830
			5	1,350
			10	1,600
			25	2,440
			50	2,990
			100	3,880
			200	4,780
			500	5,630
Berryessa Creek at Yosemite Rd	9.20	B15	2	620
			5	990
			10	1,170
			25	1,770
			50	2,200
			100	2,910
			200	3,580
			500	4,290
Berryessa Creek at Landess Road	8.91	B13	2	610
			5	960
			10	1,120
			25	1,620
			50	2,020
			100	2,780
			200	3,490
			500	4,200
Berryessa Creek Below Confluence with Sierra Creek (Morrill Rd)	7.70	B11	2	470
			5	710
			10	830
			25	1,260
			50	1,630
			100	2,140
			200	2,660
			500	3,140
Berryessa Creek Below Confluence with Crosley Creek	5.75	B9	2	300
			5	500
			10	700
			25	1,000
			50	1,340
			100	1,740
			200	2,220
			500	2,600

Revised Table 15a. 24-Hour Storm Flood Frequency Quantiles (Cont.)

Location	**Drainage Area** (sq mi)	**HMS Node**	**Recurrence Interval** (yrs)	**Future Conditions** (cfs)[1]
Berryessa Creek Below Confluence with Sweigert Creek	4.91	B7	2	260
			5	450
			10	600
			25	890
			50	1,180
			100	1,530
			200	1,960
			500	2,300
Berryessa Creek at Old Piedmont Road	4.40	B5	2	240
			5	420
			10	560
			25	830
			50	1,090
			100	1,430
			200	1,820
			500	2,130
Piedmont Creek at Mouth		RP4-B17	2	230
			5	370
			10	440
			25	720
			50	960
			100	1,150
			200	1,380
			500	1,580
Arroyo de Los Coches at Mouth		RCL8-B17a	2	250
			5	410
			10	540
			25	780
			50	990
			100	1,270
			200	1,600
			500	1,860
Tularcitos Creek at Mouth		RT4-T3	2	190
			5	290
			10	360
			25	530
			50	650
			100	830
			200	970
			500	1,150
Calera Creek at Mouth		CAL3	2	170
			5	280
			10	360
			25	510
			50	650
			100	840
			200	1,070
			500	1,230

[1] Revised flows as of October 2006

Errata

While translating the April 2003 HEC-HMS model from version 2.1.1 to version 3.0.1 a few minor typographical errors in the Tables from the April 2003 report were found. Corrections to these tables are shown below.

Correction to Table 6. 24-Hour Storm Loss Rate Parameters

The initial loss rate for sub-basins B2, B4, B6, B8, B10, CAL2, CR2, LC2, LC4, LC6, P2, SIE2, ST2, SW2, and T2 in the 200-500 year HEC-HMS basin models was 0.27 inches. Table 6 in the report erroneously listed these as 0.24 inches. However, these parameters were entered correctly in the original 2003 model.

Correction to Table 9. Percent Impervious and Pervious Area by Subbasin

Table 9 should have listed the following values for future land-use condition impervious and pervious area percentages for the LP8, LP16, and ST2 sub-basins.

Future Condition		
Subbasin	**% Impervious**	**% Pervious**
LP8	31.0	69.0
LP16	71.0	29.0
ST2	14.0	86.0

Correction to Table 12. Stream Reach Characteristics and Muskingum Routing Parameters

The Muskingum K, Muskingum X, and N steps columns in the 2003 report were entered out of order. These parameters were entered correctly in the original 2003 model. The corrected table is shown below.

Reach Name	Reach Length	Reach Slope	Velocity	Muskingum K	Muskingum X	N Steps
RLC1-LC2	1.204	0.0408	6.0	0.29	0.25	2
RB1-B3	1.473	0.0831	6.0	0.36	0.25	3
RB3-B5	1.872	0.0544	6.0	0.46	0.25	3

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1 INTRODUCTION

1.1 General Information

This report documents the hydrologic analysis of the Berryessa Creek drainage basin in Santa Clara County, California, as performed by northwest hydraulic consultants, inc. (**nhc**). The objectives of this report are to summarize the analyses undertaken, document data and key assumptions, and present the results of the hydrologic study. The study was conducted to estimate peak flow discharges at key locations along Berryessa Creek for use in the design and evaluation of a flood damage reduction project for the reach of Berryessa Creek between Calaveras Boulevard and Lower Penitencia Creek. This study was undertaken at the request of the Santa Clara Valley Water District (SCVWD) in accordance with their need to provide flood control improvements.

This study consisted of the development and application of an HEC-HMS rainfall-runoff hydrologic simulation model of the Berryessa Creek drainage basin. Instantaneous peak discharges corresponding to 50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2-percent chance exceedance events were calculated at key index points within the study area of Berryessa Creek. These discharges will be used in the hydraulic analysis and design of the flood impact reduction projects. Peak discharges were calculated for both existing and anticipated future (full build-out) land-use conditions based on available land use information provided by the SCVWD.

The hydrologic data developed in this study will also be used by the Sacramento District of the Corps of Engineers (COE) for a flood damage reduction project on Berryessa Creek between Old Piedmont Road and Calaveras Boulevard. To accommodate requirements of the Corps of Engineers' project, an extended duration storm was developed and simulated to estimate runoff volumes for longer duration storm events.

This report provides information on the hydrometeorologic and geographic characteristics of the watershed and their effect on runoff and flooding potential. This report also describes the development of the HEC-HMS model, derivation of input data used in the model, key assumptions, and results from the hydrologic investigation. The data, assumptions, methods, and results presented herein have been thoroughly reviewed and accepted by the SCVWD and the COE.

1.2 Study Area

The Berryessa Creek drainage basin is located northeast of San Jose, California, as shown in Figure 1. The basin is located immediately north of the Upper Penitencia Creek drainage basin. Berryessa Creek flows west out of the foothills of the Diablo Range in Santa Clara County, then continues in a westerly direction through the southern portion of the City of Milpitas. The creek flows north through the City of Milpitas, joining Lower Penitencia Creek approximately 4,400 feet above the confluence of Lower Penitencia Creek and Coyote Creek. Coyote Creek enters the San Francisco Bay approximately 10 miles northwest of San Jose. The Berryessa Creek drainage basin encompasses 22.4 square miles of rural and urban land upstream of its confluence with Lower Penitencia Creek. The Lower Penitencia Creek basin, at its confluence with Coyote Creek, drains approximately 29 square miles. Ground elevations in the basin range from 2,600 feet above mean sea level (msl) in the foothills to 10 feet above msl at the confluence with Coyote Creek.

1.3 Climate

The Berryessa Creek basin lies within a region that has a mild Mediterranean climate. This is characterized by almost all of the rainfall occurring between November and early April, and mild temperatures that are rarely below freezing during the winter and seldom exceed 100 degrees during the summer. Long term temperature and precipitation data are available from the National Weather Service's San Jose Climatological Station 7821, located approximately 5.5 miles south of the Berryessa Creek basin. The mean annual temperature at this station is 60.1 degrees Fahrenheit (ºF). The average monthly temperature ranges from a low of 49.1ºF in December to a high of 69.5ºF in July. The mean annual precipitation at the San Jose station is 14.42 inches, with the month of June being the driest and January being the wettest. Climatological data for this station are summarized in Table 1 (Owenby and Ezell, 1992).

Precipitation events in the Berryessa Creek basin are predominately a result of either orographic or cyclonic effects. Orographic storms occur when moist air rises, cools and condenses as it moves over the Diablo Hills and mountain range. Cyclonic storms are the result of air masses moving from regions of higher barometric pressure to lower pressure zones. Additionally, cyclonic storms may be the result of frontal activity. Broad bands of low intensity rainfall are associated with warm fronts, while higher intensity rainfall is associated with cold fronts (Schaaf and Wheeler, 2001). Cloudburst storms or convective type storms with intense central rain and limited spatial distribution rarely occur.

1.4 Flood Characteristics

Flooding resulting from intense widespread precipitation over the Santa Clara Valley and Diablo Range typically occurs between November and April. These floods are characterized by high peak flows of moderate durations which are intensified when the ground is saturated. The storms that lead to flooding are typically cyclonic storms originating in the Pacific Ocean.

Runoff from urban areas in the basin is conveyed to main channels by municipal storm drainage systems. Streets serve as temporary collection basins upstream of the storm drainage systems. During extreme rainfall events, when flows exceed the capacity of the storm drainage system, excess water flows overland and down the streets. Localized flooding in the urban portions of the basin results from undersized storm drains and sediment or debris clogging the storm drain inlets. More damaging flooding is induced by inadequate channel capacities in major collection and conveyance systems such as Berryessa Creek.

2 HYDROLOGIC MODEL

2.1 Overview

Observed stream flow data for Berryessa Creek are minimal and out of date, and it is therefore necessary to generate synthetic design flow data using a hydrologic model. The hydrologic model developed for this study includes the entire Berryessa Creek watershed. The hydrologic model also includes the Lower Penitencia Creek watershed, even though this project does not include the design of flood control facilities on Lower Penitencia Creek. Lower Penitencia Creek was included in the model because flow in Lower Penitencia Creek will affect the hydraulics of Berryessa Creek, and thus may affect the design of flood control measures on Berryessa Creek.

The model selected to simulate the rainfall-runoff processes for Berryessa Creek is the U.S. Army Corps of Engineers' Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS). Within HEC-HMS, runoff is simulated for a large basin by evaluating runoff from a number of smaller subbasins. The runoff from these smaller subbasins is routed to the main channel, combined with flow from other subbasins, and routed down the main channel. This approach can provide a reasonably accurate, process based, representation of flow paths, storage, and attenuation across the watershed.

HEC-HMS requires the input of data to define the estimation of rainfall excess, the transformation of this excess into direct runoff within each subbasin, the estimation of baseflow, and the routing of computed flood hydrographs in the main channel. The following subsections describe the modeling methods used and the process in which the input data were developed.

2.2 Subbasin Delineation

The Berryessa Creek and Lower Penitencia Creek drainage basins were divided into two general categories: urbanized and rural. Descriptions of these categories are provided below.

1. *Urban Areas*: The majority of urban development in the Berryessa Creek basin is located in the lower elevation and western portion of the basin within the Milpitas city limits. Essentially all of the Lower Penitencia Creek basin is urbanized. Single-family housing, apartment complexes, shopping centers, schools, and industrial areas create a high percentage of impervious cover. All conveyance features are improved to some degree. These features vary from slightly improved open channels to complex underground storm drain systems. Urban areas are characterized by ground slopes of about 1 percent (0.01 $^{ft}/_{ft}$). Soils maps of the area (SCS, 1964) indicate that younger clays are interspersed with smaller amounts of old San Francisco Bay mud in the vicinity of the creeks. Much of the soil in the urban area is either clay or clayey loam and is thus associated with very low infiltration rates when wetted, resulting in high runoff potential. Closer to Coyote Creek the soil becomes loamier, and is thus characterized by better infiltration characteristics and moderate to high runoff potential.

2. *Rural Areas*: Under existing conditions, the higher elevation portions of the Berryessa Creek basin, representing the foothills area east of Old Piedmont Road, are either undeveloped or sparsely developed. The tributary channels in the upper basin wind through gently rolling grass-covered hills. The upper basin is characterized by slopes of 5 to 6 percent (0.05 to 0.06 $^{ft}/_{ft}$) with minimal impervious cover. The soils are shallow with high clay content. Stream channels are commonly flanked by brush and deciduous trees. The width of the riparian zone along these channels varies from several hundred yards in the channels of Berryessa Creek and Arroyo de Los Coches, to a few yards on the smaller tributaries.

For hydrologic modeling purposes, the study area was divided into 39 subbasins (see Figure 2). The rural zone was divided into 16 subbasins, representing the foothill basins of Berryessa Creek, Piedmont Creek, Sierra Creek, Crosley Creek, Sweigert Creek, Arroyo de Los Coches, Tularcitos Creek, and Calera Creek. The lower (urban) region was divided into 23 subbasins, each representing to an area of similar flow patterns and urbanization. The drainage divides defining these subbasins were established by referencing U.S. Geological Survey quadrangles, county storm drainage design maps, field inspections, the Milpitas Master Drainage Plan and consultations with City staff familiar with the area (Schaaf and Wheeler, 2001; Delorme, 2000).

2.3 Precipitation Data

Rainfall hyetographs were developed for the 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year return periods. At the direction of the Santa Clara Valley Water District, all of the modeled storm events use the same rainfall distribution. The prescribed distribution is the 24-hour rainfall pattern developed by the COE for their 1982 hydrologic study of Berryessa Creek (USACE 1982). This unitless distribution is shown in Table 2. The hyetograph for a particular storm event was determined by multiplying the percentage of rainfall in each time step by the total rainfall depth for the event.

The 24-hour rainfall totals for the 2- through 100-year storms were derived using the NOAA Atlas 2 (NOAA 1973). As a result of orographic effects, precipitation is significantly greater at the higher elevations in the basin than at lower elevations. To accurately simulate the spatial variability of precipitation during a large event, the study area subbasins were grouped into three representative precipitation regions, labeled Urban Penitencia, Urban Berryessa and Foothills (Table 3). Individual subbasins were grouped into one of these three regions based on the NOAA Atlas 2 isopluvials in or adjacent to the subbasin. The location of the three precipitation regions is shown in Figure 3. To determine precipitation depth, a map of the study area and delineated subbasins was digitally overlaid onto 2- through 100-year isopluvial maps from NOAA Atlas 2, and average precipitation depths were determined for each of the three regions.

Because the NOAA Atlas does not include isopluvial maps for 200- and 500-year events, the precipitation depths for these return periods were extrapolated from a log-normal plot of the 2- through 100-year NOAA Atlas 2 rainfall depths (see Figure 4). Other statistical distributions were examined, but the log-normal distribution shown in Figure 4 provided the best fit to the data.

Depth-area reduction factors were applied to the 24-hour rainfall volumes to account for the size of the drainage basins in the study area. The values of the depth-area reduction factors were determined from Figure 14 in the NOAA Atlas 2 and applied to the storm precipitation totals. The reduction factor for the Urban Berryessa and the Foothills regions is 0.972. This value corresponds to a storm centered over the entire 22.36 mi^2 Berryessa Creek drainage basin, and the value is obtained directly from the 24-hour duration depth-area curve in Figure 14.

The depth-area reduction factor for the Urban Penitencia region is 0.943, which was derived through an indirect procedure. The value for the Urban Penitencia region corresponds to the reduction factor necessary to generate a "concurrent storm" over the 5.88 mi^2 Lower Penitencia Creek drainage basin. The concurrent storm is the rainfall that occurs over the Lower Penitencia Creek drainage basin while a storm of a specific recurrence interval is occurring over the Berryessa Creek drainage basin. This approach was justified because Lower Penitencia Creek was not being analyzed for the purpose of designing flood control features along the Lower Penitencia Creek channel, but rather to establish boundary conditions at the downstream end of Berryessa Creek. Therefore, flow rates on Lower Penitencia Creek during a concurrent storm are theoretically more applicable for the design of flood protection features on Berryessa Creek than peak flow rates associated with an equal recurrence interval storm over the Lower Penitencia Creek basin. The reduction factor for the Lower Penitencia Creek basin was obtained by first determining the depth-area reduction factor for the 28.24 mi^2 combined Berryessa Creek and Lower Penitencia Creek drainage basin (0.966 from NOAA Atlas 2 Figure 14, 24-hour duration curve). The final step was to calculate the value that must be applied to only the 5.88 mi^2 Lower Penitencia Creek basin if 0.972 is applied to the 22.36 mi^2 Berryessa Creek basin, and 0.966 is the appropriate depth area reduction value to apply to the 28.24mi^2 combined basin. This is represented in equation form as:

$$(0.966)(28.24) = (0.972)(22.36) + x(5.88)$$

The rainfall depths for each region and return period, with depth-area reduction factors applied, are presented in Table 4.

At the request of the Corps of Engineers, 1-, 6-, 12- and 24-hour rainfall depths used in this study were compared to data from precipitation gages located near the study area. This comparison is shown in Table 5. There are four precipitation gages within the drainage basin boundaries or near the study area: San Jose, Penitencia Water Treatment Plant, Curtner Ranch and Haskins Ranch. The depths reported for the gage data were developed by Jim Goodridge, former California State Climatologist with the Department of Water Resources. The San Jose gage had a record length of approximately 90 years, while the other three gages had record lengths between 22 and 29 years. Comparison of these data does not show a strong correlation. For all recurrence intervals, the gage data has greater depths for shorter duration storms such as the 1-hour storm, whereas the NOAA data has greater depths for longer duration storms such as the 24-hour storm. Differences between the data derived from the NOAA Atlas 2 and the data from local gages is possibly a result of the difference in the time span over which the data were collected, or possibly the procedures used by NOAA to develop the Atlas 2 isopluvial mapping. The SCVWD and COE selected to use the NOAA Atlas 2 data for this study rather than the local gage data.

2.4 Precipitation Excess

For this analysis, rainfall losses due to interception, depression storage, and infiltration, are estimated in the HEC-HMS model using the initial and constant loss rate method. With this method, interception and depression storage are represented with an initial loss. Precipitation on pervious surfaces only contributes to runoff after a volume equal to this initial loss has occurred since the beginning of the storm event, and thereafter only the amount of rainfall in excess of the constant loss rate contributes to runoff.

Initial and constant losses were estimated based on values used by the San Francisco District, Corps of Engineers for the hydrologic evaluation of Upper Penitencia Creek (USACE 2001). Initial and constant losses were increased slightly for the rural, upland areas based on the assumption that these areas would consist of more permeable, undisturbed soils. The adopted initial losses and constant loss rates are listed in Table 6.

2.5 Baseflow

Baseflow in the study area is minimal and has a negligible effect on flood peaks. For this analysis the exponential recession model was selected for base flow simulation. This method requires the user to input an initial flow at the beginning of the storm event (typically the average annual flow in the channel), a recession constant that is dependent on the basin size and type of flow expected to contribute to base flow, and a threshold value which defines the time when the recession model defines the total flow. Three different sets of parameters were adopted based on the location of the subbasin. These parameters are listed in Table 7.

2.6 Land Use

An important consideration in the development of the HEC-HMS model is the proper classification of land cover as either pervious or impervious. Runoff rates are highly influenced by the amount of land in a basin that is impervious. Increased imperviousness, as found in urban areas, results in reduced opportunity for infiltration and thus increased runoff. Streets, gutters, rooftops, parking lots, driveways, and similar areas are considered impervious areas when determining the percentage of imperviousness in each subbasin. Hard packed soil, such as gravel roads and parking lots, are also considered as impervious for purposes of the hydrologic modeling.

Geographical Information System (GIS) mapping of existing and future land-use conditions was used to estimate percent pervious and impervious cover for each subbasin. These land-use maps, shown in Figures 5 and 6, provide a spatial representation of land-use in digital format. From these maps, the amount of each type of land-use was calculated for the individual subbasins through use of the GIS.

The existing conditions land use map was developed from several sources, including 1993 aerial photography of the City of Milpitas for the urban areas and 1970 land use delineation by the USEPA for the rural areas. After reviewing the available maps, eight general categories of land use were selected to define the varying land use in the basin. These are: Commercial/Industrial (CI), Forested (F), Grassland (GL), Park (PK), High Density Residential (R-1), Medium Density Residential (R-2), Low Density Residential (R-3), and Rural Residential (R-4). Figure 5 shows the drainage basin delineated by existing conditions land use.

Once the land use within the drainage basin was delineated, an estimate of the percent of impervious coverage that typically occurs within each specific land use category is made. This was done by estimating pervious and impervious coverage for representative areas within each land-use classification and using this to assign an impervious percentage for the classification. The amount of impervious land cover in a particular subbasin is calculated as the product of the area of each land use category within the subbasin and the percent impervious cover associated with the specific land use category. Note that modeled percent impervious areas may be lower than the actual impervious coverage in a basin. This is because some portion of impervious surfaces may discharge onto pervious areas prior to reaching a stormwater collection system (e.g., residential rooftops may drain onto splash blocks and across lawns before entering the storm drainage system). The estimated percent of effective impervious cover for each land use category is shown in Table 8. The percentage of impervious land cover for each subbasin is shown in Table 9.

A future conditions land use map of the study area (Figure 6) was developed from zoning maps included in the General Plans for the cities of Milpitas and San Jose, and Santa Clara County. The future land use conditions map represents a full build-out condition in the study area, as defined by the General Plans. Future land use conversions, such as from grassland to residential or from residential to industrial are simulated in HEC-HMS model by increasing the percentage of impervious land cover. Future conditions impervious coverage is estimated using a similar procedure as was described for the existing conditions land-use. Table 9 lists the estimated future conditions impervious areas by subbasin.

2.7 Rural Subbasin Runoff Method

The unit hydrograph method was selected as the most applicable method to transform excess precipitation to runoff in the rural portions of the Berryessa Creek basin. Representative unit hydrographs were developed using the summation-curve method (S-curve). The S-curve used to determine subbasin unit hydrographs for this study was originally developed by the Sacramento District, Corps of Engineers in their 1982 hydrologic analysis of Berryessa Creek (USACE 1982). The Corps of Engineers developed an S-curve based on reconstitution of historical events recorded at the streamflow gage on Upper Penitencia Creek. The drainage basin upstream of the Upper Penitencia Creek streamflow gage is located immediately south of the Berryessa Creek drainage basin and has generally similar topographic and land-use characteristics.

Unit hydrographs for each subbasin were developed by calculating and applying a Basin Lag or shaping factor unique to each subbasin. The San Francisco District, Corps of Engineers District-wide relationship for lag was used for this analysis and is:

$$\text{Basin Lag} = 24n((L\ L_{CA})/S^{1/2})^{0.38}$$

Where:

Lag = time from the start of the runoff to a point where 50-percent of the ultimate unit hydrograph runoff has occurred, in hours;

n = average roughness coefficient for the basin, dimensionless;

L = length (longest flow path) of the main watercourse, in miles; and

L_{CA} = length (flow path) on the main watercourse from a point closest to the center of area of the basin to the basin outlet, in miles.

S = slope of the basin, in feet/mile

Basin lag parameters used in the HEC-HMS model for each of the rural subbasins are summarized in Table 10.

2.8 Urban Subbasin Runoff Method

The kinematic wave method was used to transform excess precipitation to runoff in the urban areas within the Berryessa Creek and Lower Penitencia Creek drainage basins. The kinematic wave method was selected because there were insufficient data available to develop reliable unit hydrographs for the urban areas. The kinematic wave method is a conceptual model of watershed response. It utilizes input parameters for the drainage subbasins that can be estimated from physical basin characteristics (e.g., slope, distance, channel shape) and provides a reasonable means of transforming excess precipitation to runoff when data are not available for unit hydrograph calibration. A detailed description of this technique is contained in “Introduction and Application of Kinematic Wave Routing Techniques Used in HEC-1”, (USACE 1993).

The Kinematic Wave runoff hydrograph is shaped by routing excess precipitation through a theoretical series of open channels. First the runoff is routed through an overland flow plain (a wide shallow channel) that represents sheet flow. The runoff is then routed through sub-collector channels, which typically represent gutters, then through collector channels, which represent major ditches or storm drain pipes, to the downstream end of the subbasin. Each of these routings are calculated using the momentum and continuity equations to simulate unsteady open channel flow. The kinematic wave parameters used in the HEC-HMS model for the urban subbasins are listed in Table 11.

2.9 Rural Hillside Streamflow Routing

The Muskingum routing method was used for routing flow hydrographs through creek channel reaches in the undeveloped hillside areas. Flow velocity was initially estimated using Manning’s equation assuming the roughness coefficient (n) was equal to 0.045; however, this produced flow velocities that seemed unrealistically high. A more reasonable estimate of flow velocity for the upper basin channels of 6 feet per second was used for the Muskingum routing calculations. The Muskingum travel time, K, was estimated as the stream length divided by velocity. Lacking calibration data to accurately calculate the Muskingum X value, a value of 0.25 was assumed appropriate for the streams involved because it is the midpoint of the typical range of values. The number of calculation steps was estimated based on the travel time and a time step of fifteen minutes used in the HEC-HMS simulations. Table 12 shows the Muskingum routing parameters used in this analysis.

2.10 Urban Area Main Channel Streamflow Routing

The Modified Puls method was used to route flow hydrographs through the urban reaches of Berryessa Creek. The required storage-outflow relationship for each sub-reach was produced using an HEC-RAS model created from several existing HEC-2 models of Berryessa Creek and Lower Penitencia Creek provided by the SCVWD from work done during previous studies. The composite HEC-RAS model was modified to eliminate levee overtopping by incorporating infinitely high, vertical, frictionless walls at the top of the bank along both sides of the channel. This was done to contain all flow in the main Berryessa Creek channel, as is anticipated following completion of the proposed Berryessa Creek flood control projects.

Sub-reaches were defined between key locations, such as the confluence of tributary streams, pump station outlets, or significant road crossings. A range of flows was analyzed with the HEC-RAS model to develop storage versus discharge relationships for the hydrologic model.

The Modified Puls method was also used to create routing reaches for portions of Lower Penitencia, Tularcitos, Los Coches and Piedmont Creeks. Storage-outflow tables were developed from HEC-RAS models of each creek. The models were based on channel cross sections surveyed by **nhc** in April 2002.

Because the HEC-HMS model was assumed to include future improvements on these channels, flow in the HEC-RAS model was confined to the channel, as was done for the main stem of Berryessa Creek.

The Modified Puls routing parameters for all applicable reaches are shown in Table 13. Figure 7 is a schematic diagram of the HEC-HMS model, and shows the relative locations and connectivity of all the nodes listed in Table 13.

2.11 Pump Station Data

Levees are present along portions of Berryessa and Lower Penitencia Creeks. These levees provide flood protection, but also function as a barrier that can prevent runoff generated on the land behind the levees from flowing directly into the creeks. In these areas, runoff is routed to low elevation locations (or sumps) and pumped into the creek. There are nine pump stations in the study area which are designed to convey interior runoff to the creeks: California Circle, Minnis, Abbott, Penitencia, Berryessa, Jurgens, Wrigley-Ford, Manor and Spence Creek.

Elevation-area-outflow or storage-outflow data were input into the HEC-HMS model for each of the pump stations based on data from the City of Milpitas Master Drainage Plan (Schaaf & Wheeler, 2001) or provided by Schaaf & Wheeler. Pump station data are summarized in Table 14. Relative locations of the pump stations are shown in Figure 7.

2.12 Breakout Flow Routing

The existing Berryessa Creek channel has insufficient capacity to convey all of the flow during large storm events. When floods greater than approximately a 10-year recurrence interval occur under existing conditions, flow overtops the banks and spills onto the floodplain at some locations. This allows significant attenuation of the flood hydrograph, reducing the peak flow downstream of breakout locations. Because the purpose of the current Berryessa Creek projects is to reduce flooding by improving channel capacity, this hydrologic analysis assumed that Berryessa Creek would be able to convey all of the simulated design flows without overtopping. This is a necessary assumption to be able to predict appropriate peak flow rates for project design purposes along the entire length of the creek channel.

For the tributary channels, however, it is unknown at this time to what extent flood improvements will be implemented in the near future. At the direction of the SCVWD, this study assumed that no improvements will be made on any of the tributaries to Berryessa Creek, or on Lower Penitencia Creek. At some locations on the tributary creeks, high flows spill over the channel banks or “break out” and travel overland, either re-entering the tributary creek further downstream or flowing overland into a different subbasin. These breakout flows are simulated in the HEC-HMS model through the use of flow diversion tables inserted in the affected flow reaches. Figure 7 provides a schematic diagram of all the flow connections in the model, including breakout flows.

Several breakout flow locations were identified along tributary streams in the study area. A culvert on Piedmont Creek immediately upstream of I-680 was calculated to have a maximum capacity of 900 cfs; flows greater than 900 cfs will break out of the channel and travel north along South Park Victoria Drive, eventually entering Los Coches Creek. The culvert under I-680 on Los Coches Creek was calculated to have a maximum capacity of 800 cfs, and excess flows will break out and flow north along South Park Victoria Drive. Based on available topography and field investigations, the assumed overland flow path of the breakout flows from Los Coches Creek is west along Calaveras Boulevard. A portion of the breakout flow will enter Berryessa Creek at the Calaveras Boulevard bridge, and the remainder of the flow will continue west along the roadway, ultimately reaching the Wrigley Creek subbasin.

The reach of Tularcitos Creek downstream of I-680 was calculated to have a maximum capacity of 700 cfs. Flows greater than 700 cfs will overtop the channel and travel north. This flow will end up in subbasin B20, which is drained by the Berryessa pump station.

Other culvert crossings along the tributary creeks were evaluated, but overtopping flows in these areas were thought likely to re-enter the same creek channel immediately downstream of the crossing, thus no breakout flows are modeled in these reaches.

2.13 Subbasin Storage

At the request of the SCVWD, an investigation was performed to determine if the routing storage implicitly calculated using the kinematic wave procedure in the urban portions of the study area is reasonable. The initial concern was that the simplified approach utilized by the kinematic wave method in HEC-HMS would underestimate storage, thus decreasing flow attenuation and increasing peak flow rates. The investigation included: (1) field observations and measurements to locate areas where additional flow storage could occur, (2) manual routing calculations to determine the volume of storage in selected storm drain pipe systems and associated roadway gutters when storms of greater magnitude than the design storm event occur, and (3) calculation of storage at the upstream end of culvert crossings. The results from these calculations were compared to the volumes of storage calculated by HEC-HMS when using the kinematic wave method.

The investigation found that the kinematic wave method in HEC-HMS was adequately calculating storage for all the subbasins in the Berryessa Creek drainage basin. However, at two locations in the Lower Penitencia Creek drainage basin the investigation indicated that the kinematic wave method was potentially underestimating the amount of storage that would occur during extremely large flooding events. At these two locations in Lower Penitencia Creek, storage was added by inserting a reservoir storage table at the downstream end of the subbasins.

The first area that appeared to require additional storage was the farthest upstream subbasin of Lower Penitencia Creek (subbasin LP2). There is no open channel in this subbasin, and Lower Penitencia Creek is essentially defined by three large storm drain pipes. When the capacity of the storm drain system is exceeded, excess runoff flows north towards the open channels that are the beginning of Lower Penitencia Creek. A portion of the excess flow is likely to travel overland towards the west and pond against the Interstate 880 embankment. Because the hydraulic connections within of this subbasin during large storm events are very complex and no measured data or anecdotal observations of large flood events were available for **nhc** to use as a basis for modifying the HEC-HMS model, and because the study of this creek is only necessary to establish boundary conditions for the proposed project on Berryessa Creek, the SCVWD decided to implement a simplifying assumption rather than perform a detailed interior drainage analysis. The assumption developed by the SCVWD and implemented in the HEC-HMS model was that the maximum rate of flow from the LP2 subbasin would be limited to the capacity of the storm drain system operating with one foot of head on the farthest downstream run of pipe in the storm drain system. Calculations based on storm drain pipe data provided by City of San Jose drainage maps indicated that this assumption limits the flow from LP2 to 800 cfs. In the HEC-HMS model, when runoff from the LP2 subbasin exceeds 800 cfs, the excess flow is temporarily placed in storage until the runoff from the subbasin drops below 800 cfs, at which time the volume in storage is routed out of the subbasin at a maximum rate of 800 cfs. In the HEC-HMS model there is no limit to the volume of storage that is available to maintain a maximum outflow rate of 800 cfs for any flooding event, and **nhc** did not investigate if the amount of storage used by the HEC-HMS model is physically reasonable for the LP2 subbasin.

The other area that appeared to require additional storage was subbasin LP4, specifically the area along Lower Penitencia Creek between Great Mall Parkway and Calaveras Boulevard. When flow in the creek exceeds 1200 cfs, the hydraulic capacity of a series of culverts in this area is exceeded. The excess flow overtops the channel and would be expected to pond in the topographically flat area east of Lower Penitencia Creek and west of Wrigley-Ford Creek. To simulate this storage, a reservoir storage table was added to the HEC-HMS model at the downstream end of subbasin LP4. When flow in the main channel of Lower Penitencia Creek through LP4 exceeds 1,200 cfs, a portion of the excess flow is temporarily placed in storage. The volume of available storage was determined from topography maps of the Lower Penitencia Creek floodplain. The relationship of the amount of runoff in storage to the flow in the main channel of the creek was based on the hydraulic characteristics of the limiting culverts and the available storage.

3 RESULTS – PEAK FLOW ANALYSIS

3.1 Peak Flow Frequency Curves

Eight storm events, corresponding to the 50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2-percent chance exceedance events (2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year recurrence intervals) in the Berryessa Creek drainage basin, were simulated using the HEC-HMS model. Peak discharges were calculated for both existing and future (full build-out) land-use conditions. Existing conditions 10-, 50-, 100- and 500-year flows at four key locations are summarized in the table below. A complete listing of results from the HEC-HMS model are presented in Table 15. Hydrographs and cumulative flow volumes for the 1- percent chance exceedance event (100-year event) at Old Piedmont Road and Calaveras Boulevard are presented in Figures 8 and 9.

Berryessa Creek - Existing Conditions				
	Peak Flow (cfs)			
Location	10-year	50-year	100-year	500-year
at Old Piedmont Road	530	1,060	1,400	2,110
at Calaveras Boulevard	1,990	3,650	4,690	6,430
at North Abel Street	2,730	4,620	5,580	8,140
at Downstream End of Berryessa Creek	3,050	5,270	6,430	9,180

3.2 Comparison of Peak Flows with Other Data and Studies

To investigate the reasonableness of peak flows calculated with the HEC-HMS model, the results from the model at selected locations were compared to results from regression equations, streamflow gage station flood-frequency analyses, and with previous hydrology studies of the Berryessa Creek basin.

3.2.1 Regression Analysis

The 100-year peak flow rate was calculated for subbasins B2 and CAL2, and for Berryessa Creek at Old Piedmont Road (HEC-HMS index point B5) using the regression equations from the SCVWD Hydrology Procedures manual (Wang, et. al., 1998). These three subbasins were selected to represent a sampling of the rural areas within the Berryessa Creek drainage basin. The regression equations provided in the Hydrology Procedures manual were developed for application to streams with minimal development, and the selection of these three subbasins is consistent with this requirement.

Results from this comparison are presented in Table 16. In addition to the 100-year peak flow rate, confidence intervals corresponding to a 90% confidence level were calculated for each of the basins. This comparison shows that the HEC-HMS model generates a 100-year peak discharge that is within the 90% confidence interval of the regression equations for the two smaller subbasins, and slightly greater than the upper 90% confidence limit for the Berryessa Creek at Old Piedmont Road subbasin. This indicates that there is some agreement between the HEC-HMS model results and the regression equations, although not an exact match.

3.2.2 Streamflow Gage Data Flood Frequency Analysis

Further evaluation of the HEC-HMS model was accomplished by comparing peak flows calculated by the model to peak flows generated from frequency analysis of available streamflow gage data. Two sets of streamflow gage data were used for this analysis. The first is from a streamflow gage that was operated for a short period of time by the SCVWD on Berryessa Creek at Calaveras Boulevard. The other is from a streamflow gage operated by the USGS on Upper Penitencia Creek (USGS station 11172100) located at the downstream end of the rural portion of that drainage basin. A statistical analysis was performed on both data sets using HEC-FFA (USACE 1992), which is a flood frequency analysis program developed by the Corps of Engineers. The statistical analysis fit a log-Pearson Type III distribution to the data sets.

The Berryessa Creek at Calaveras Boulevard gage represents the most directly useful data set; however, the data set is relatively short (gage operation from 1968 through 1983) and much of the drainage basin has been urbanized since the gage was discontinued. In addition, the Berryessa Creek channel is unable to convey all of the flow during an extremely large flood event. If an extreme event occurred during the period of gage operation, a portion of the flow would have spilled out of the channel onto the floodplain, which would have attenuated the peak, thus reducing the peak flow recorded by the gage. However, the HEC-HMS model for this project assumes flood improvements are in place and all flow is contained within the main channel, so there is no peak attenuation from floodplain storage. With this in mind, the gage information shows a surprisingly good correlation to calculated flows from HEC-HMS.

The Upper Penitencia Creek gage is useful because the drainage basin is adjacent to Berryessa Creek and should have similar rainfall and soil characteristics. The Upper Penitencia Creek basin is slightly larger than the rural portion of the Berryessa Creek drainage basin, and includes a reservoir. Both of these are likely to lead to lowering the peak flow rate in Upper Penitencia Creek. It is useful to compare peak flows from this gage data on a unit basis (cfs/mi^2) to the HEC-HMS simulated flow rates for rural subbasins in Berryessa Creek. As expected, the unit discharge from the Upper Penitencia Creek gage station is lower than the calculated flows in the rural subbasins of Berryessa Creek. However, when the size of the gaged basin and the presence of reservoir storage in the gaged basin are considered, the flows generated from the HEC-HMS model show reasonable agreement to the gaged flows.

A summary of the flood frequency analysis for the two gages is provided in Table 17. These values can be compared to the HEC-HMS model results in Table 15.

3.2.3 Other Berryessa Creek Studies

The Sacramento District of the Corps of Engineers and the SCVWD have previously conducted hydrology studies of Berryessa Creek (USACE 1982, Saah 1978). At three locations along Berryessa Creek, the existing conditions 10- and 100-year recurrence interval peak flows from the present study were compared to the flows predicted in the previous studies. There is a strong correlation between the results of the HEC-HMS model developed for this study and the results of the previous studies. The comparison of flows is shown in Table 18.

4 EXTENDED DURATION STORM

4.1 Background

The analysis discussed in the preceding sections comprised a peak flow hydrology study that was conducted in support of the SCVWD's Berryessa Creek Levees Project. That analysis used a synthetic 24-hour rainfall hyetograph that was developed by nesting maximum rainfall amounts from a series of storm durations (1-, 2-, 3-, 6-, 12-, and 24-hour rainfalls) to create a single hyetograph. This procedure is effective in evaluating the drainage basin runoff response to various storm durations using a single hyetograph. This procedure produces the peak flow for a selected recurrence interval regardless of the critical storm duration at any particular point in the basin; for instance, whether the peak flow is generated by the 1-hour intensity or the 12-hour intensity. This technique is ideal for design considerations for the Berryessa Creek Levees Project because the project is focused on improving flood flow conveyance in the Berryessa Creek channel, which requires an assessment of the peak flow rates for specific recurrence interval storms, but is not concerned with the associated total runoff volumes.

The COE is also evaluating flood improvement opportunities on Berryessa Creek. The COE study is focused on the reach of the creek between Old Piedmont Road and Calaveras Boulevard, immediately upstream of the SCVWD project reach. Peak flow rates must be considered for design of channel improvements; additionally, the COE project is analyzing the effects of floodplain storage for the existing channel and floodplain configurations. The COE may also evaluate opportunities for detention storage of flood flows as a method to reduce flood impacts. While the hydrologic analysis performed for the SCVWD project produces reasonable estimates of peak discharges, the procedure does not produce reliable estimates of runoff volumes for long duration storms. In particular, the short duration storm used in the SCVWD analysis may not be adequate to evaluation of volume dependant issues. Determining reasonable volumes for longer duration storms is necessary for the evaluation of floodplain storage and detention basin design. Because of these additional data needs for the COE project, the COE requested that the present hydrologic analysis include the evaluation of an extended duration storm for the Berryessa Creek basin.

The COE selected a 72-hour storm as the most appropriate extended duration event. Eight storms were developed and simulated corresponding to the 50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2-percent chance exceedence events (2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year events) using HEC-HMS. The model developed for the 24-hour peak flow analysis provided the basis for the 72-hour storm model. The hyetograph (discussed in Section 4.2) and the constant soil loss rates (discussed in section 4.3) are the only data in the 24-hour model that were modified for application to the 72-hour storm analysis.

4.2 Hyetograph Development

A 72-hour rainfall distribution was developed by the COE and provided to **nhc** for use in the HEC-HMS model. The distribution is a composite of several large rainfall events recorded at precipitation gages near the Berryessa Creek drainage basin. The distribution is not based on any single storm, but rather has the characteristics of extended duration storms that were identified in the gaged data. The COE distribution consists of an hourly time series, with percent of total storm volume provided for each time step. This unitless distribution is presented in Table 19.

For the 24-hour storm analysis, rainfall volumes were obtained from the NOAA Atlas 2. However, the NOAA Atlas 2 does not include 72-hour rainfall depths so an alternative method was required for the extended duration hydrologic analysis. The COE analyzed regional precipitation gage records to determine an appropriate volume for a 72-hour storm. An evaluation of gage data was previously prepared by Jim Goodridge, a former State Climatologist for the Department of Water Resources. The COE determined that for the study location, the 100-year 72-hour storm depth is approximately equal to

1.69 times the 24-hour storm depth. To verify the appropriateness of this relationship to other recurrence intervals, **nhc** compared the 24- and 72-hour depths for the 2-, 5-, 10-, 25-, 50-, 200- and 500-year storms at precipitation gages relatively near the Berryessa Creek study area. The data comparison indicated that the 72-hour depth is generally 1.69 times greater than the 24-hour depth for all recurrence intervals. Therefore, **nhc** multiplied the 24-hour storm depths from NOAA Atlas 2 by 1.69 to generate 72-hour storm totals. The same basin area reduction factors discussed in Section 2.3 were applied to the 72-hour depths. The 72-hour rainfall depths used in the HEC-HMS model are shown in Table 20. It should be noted that the 72-hour rainfall depths used in this analysis are significantly greater than values generated by the statistical analysis of measured data for any of the four nearby precipitation gages, and significantly greater than the largest 72-hour rainfall amount measured during the period of record at any of the gages. However, the 72-hour rainfall depth used in the analysis has a corresponding relationship to the NOAA data used in the 24-hour analysis.

A set of 72-hour rainfall hyetographs were then developed by applying the 72-hour storm volumes for each recurrence interval to the unitless rainfall distribution provided by the COE.

4.3 Constant Loss Rates

After reviewing preliminary results from the 72-hour storm model, the COE concluded that the soil constant loss rates used in the model were too high for an extended duration storm and the model was not generating as large of a total volume of runoff as the COE expected. The COE determined that while the soil loss rates in the 24-hour storm model were appropriate for the shorter duration storms that control instantaneous peak flows, the soil was more likely to be saturated and have a lower constant loss rate during a long duration storm. Using data from streamflow gages on nearby creeks, the COE developed a ratio between peak flow and mean daily flow for the 50-year and 100-year floods. The COE determined a ratio of less than or equal to 0.20 should occur when comparing the 100-year peak flow (regardless of the storm duration that caused the peak flow) to the mean daily flow resulting from the 100-year 72-hour precipitation.

To achieve the desired ratio, **nhc** applied the 100-year 72-hour precipitation to the HEC-HMS model and modified the constant loss rates for the subbasins upstream of Old Piedmont Road on Berryessa Creek until the average daily runoff from the 100-year 72-hour storm divided by the peak flow for the 100-year return frequency at Old Piedmont Road (see Table 15) was 0.20 or less. When the target ratio for flows was achieved at Old Piedmont Road, the amount of change applied to the constant loss rates in the subbasins upstream of Old Piedmont Road was applied to all other subbasins in the HEC-HMS model. The values for the soil loss rates for the 72-hour storm model are listed in Table 21.

4.4 Results

Existing and future conditions flows were simulated for the 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year 72-hour events using the HEC-HMS model. To be useful for hydraulic analysis and design, the results of the extended duration storm analysis need to include the entire runoff hydrograph. Inclusion of printed tables of that information, for all recurrence interval storms and all key index points is not practical in this report. However, hydrographs at two key locations, Old Piedmont Road and Calaveras Boulevard, are presented in Figures 10 and 11. Cumulative flow volumes are also shown in these figures. A copy of the HEC-HMS model or the associated DSS database file is necessary to obtain hydrographs at other locations and for other recurrence interval storms.

Existing conditions peak flow rates from the 72-hour storm for key recurrence intervals and locations are summarized in the following table.

Berryessa Creek – Existing Conditions, 72-hour Duration Storm				
	Peak Flow (cfs)			
Location	10-year	50-year	100-year	500-year
Old Piedmont Road	240	1,020	1,310	1,870
Calaveras Boulevard	1,720	3,590	4,250	5,900
North Abel Street	2,390	4,640	5,440	7,190
at Downstream End of Berryessa Creek	2,720	5,260	6,250	8,270

Existing conditions total runoff volumes for the same recurrence intervals and locations are summarized in the following table.

Berryessa Creek – Existing Conditions, 72-hour Duration Storm				
	Runoff Volume (ac-ft)			
Location	10-year	50-year	100-year	500-year
at Old Piedmont Road	170	1,180	1,650	2,760
at Calaveras Boulevard	1,720	4,670	6,020	9,220
at North Abel Street	2,610	6,300	8,070	12,300
at Downstream End of Berryessa Creek	2,720	7,130	9,180	14,100

5 SUMMARY

The hydrologic analysis of the Berryessa Creek drainage basin discussed in this report was performed to provide the flow data necessary for the design of the SCVWD's Berryessa Creek Levees Project. Results of this analysis can be used as input to hydraulic and sediment transport models to estimate hydraulic characteristics of Berryessa Creek to support the design of levee improvements and other flood control features.

This hydrologic analysis was performed by northwest hydraulic consultants, as a subconsultant to Winzler & Kelley, for the Santa Clara Valley Water District, Coyote Creek Watershed Program. The analysis follows guidelines and procedures established by the SCVWD, with additional input and review provided by the Sacramento District Corps of Engineers. To the maximum extent practical, this analysis uses established hydrologic and engineering guidelines and methodologies. Due to the complexity of the system under study and lack of measured data for hydrologic model calibration, this analysis required several simplifying assumptions to be made. As noted in the text of this report, the SCVWD and the COE provided significant direction with regard to the data and assumptions used in this analysis.

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Table 1. Climate Data at San Jose Gage Station (NOAA Gage 7821, 1961 to 1990)

	Temperature (^{o}F)				Precipitation (in.)	
	Ave Min	Ave Max	Mean	Median	Mean	Median
Annual	49.1	71.0	60.1	60.1	14.42	14.26
January	40.6	58.2	49.4	49.1	2.78	2.27
February	43.9	62.5	53.2	53.3	2.16	1.37
March	45.4	65.3	55.4	55.2	2.58	2.04
April	47.1	70.0	58.6	58.0	1.17	0.64
May	50.9	74.6	62.7	63.2	0.26	0.07
June	54.6	79.6	67.1	67.2	0.05	0.00
July	56.6	82.4	69.5	69.3	0.06	0.00
August	56.8	82.0	69.4	69.6	0.12	0.00
September	55.8	80.6	68.2	67.8	0.24	0.03
October	51.6	74.5	63.1	63.3	0.90	0.70
November	45.5	64.4	55.0	55.4	2.11	1.57
December	40.7	57.5	49.1	49.1	1.99	2.00

Table 2. 24-Hour Rainfall Distribution

Time (hrs)	% of Storm Total	Time (hrs)	% of Storm Total	Time (hrs)	% of Storm Total	Time (hrs)	% of Storm Total
0.25	0.5	6.25	0.7	12.25	1.2	18.25	1.1
0.50	0.5	6.50	0.7	12.50	1.2	18.50	1.1
0.75	0.6	6.75	0.7	12.75	1.2	18.75	1.1
1.00	0.6	7.00	0.8	13.00	1.4	19.00	1.1
1.25	0.5	7.25	0.7	13.25	1.5	19.25	1.1
1.50	0.5	7.50	0.7	13.50	1.5	19.50	1.1
1.75	0.6	7.75	0.8	13.75	1.6	19.75	1.0
2.00	0.6	8.00	0.9	14.00	1.6	20.00	1.0
2.25	0.5	8.25	1.0	14.25	8.6	20.25	0.7
2.50	0.6	8.50	1.1	14.50	3.1	20.50	0.7
2.75	0.6	8.75	1.1	14.75	2.6	20.75	0.7
3.00	0.6	9.00	1.1	15.00	1.9	21.00	0.7
3.25	0.6	9.25	1.1	15.25	1.7	21.25	0.7
3.50	0.6	9.50	1.1	15.50	1.6	21.50	0.7
3.75	0.6	9.75	1.1	15.75	1.4	21.75	0.7
4.00	0.6	10.00	1.1	16.00	1.4	22.00	0.6
4.25	0.6	10.25	1.1	16.25	1.2	22.25	0.7
4.50	0.6	10.50	1.1	16.50	1.2	22.50	0.6
4.75	0.6	10.75	1.1	16.75	1.2	22.75	0.6
5.00	0.7	11.00	1.2	17.00	1.2	23.00	0.6
5.25	0.7	11.25	1.1	17.25	1.2	23.25	0.6
5.50	0.7	11.50	1.1	17.50	1.2	23.50	0.6
5.75	0.7	11.75	1.2	17.75	1.1	23.75	0.6
6.00	0.7	12.00	1.2	18.00	1.1	24.00	0.5

Source: 1982 USACE, *Hydrology Office Report, Berryessa Creek*

Table 3. Subbasins in Precipitation Regions

Urban Penitencia	Urban Berryessa	Foothills
CAL8	B12	B2
LP2	B14	B4
LP4	B16	B6
LP6	B18	B8
LP8	B20	B10
LP10	CAL4	CAL2
LP12	CAL6	CR2
LP14	CR4	LC2
LP16	F2	LC4
	LC8	LC6
	P4	P2
	SIE4	SIE2
	SW4	ST2
	T4	SW2
	W2	T2

Table 4. 24-Hour Precipitation Totals

Region	24-Hour Precipitation Depth (inches)							
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr	500-yr
Upper Penitencia	2.07	2.83	3.21	4.15	4.53	5.23	5.72	6.46
Upper Berryessa	2.33	3.21	3.60	4.76	5.25	6.03	6.62	7.48
Foothills	2.62	3.60	4.28	5.44	5.83	6.89	7.52	8.51

Table 5. Comparison of NOAA Data and Precipitation Gage Data (depths in inches)

2-year	NOAA Data			Precipitation Data			
	Urb Pen	Urb Berry	Foothills	San Jose	Pen WTP	Curtner R	Haskins R
1-Hour	0.34	0.38	0.43	0.42	0.45	0.49	0.49
6-Hour	0.89	1.01	1.13	0.92	0.97	1.02	1.20
12-Hour	1.44	1.62	1.82	1.19	1.24	1.27	1.58
24-Hour	2.07	2.33	2.62	1.52	1.42	1.53	1.98
5-year	NOAA Data			Precipitation Data			
	Urb Pen	Urb Berry	Foothills	San Jose	Pen WTP	Curtner R	Haskins R
1-Hour	0.46	0.52	0.58	0.59	0.63	0.69	0.70
6-Hour	1.22	1.38	1.55	1.30	1.37	1.44	1.69
12-Hour	1.96	2.22	2.49	1.68	1.75	1.79	2.23
24-Hour	2.83	3.21	3.60	2.14	2.00	2.16	2.79

Table 5. Comparison of NOAA Data and Precipitation Gage Data (Cont.)

10-year	NOAA Data			Precipitation Data			
	Urb Pen	Urb Berry	Foothills	San Jose	Pen WTP	Curtner R	Haskins R
1-Hour	0.52	0.58	0.69	0.71	0.75	0.82	0.83
6-Hour	1.38	1.55	1.84	1.55	1.64	1.72	2.01
12-Hour	2.22	2.49	2.96	2.01	2.09	2.14	2.66
24-Hour	3.21	3.60	4.28	2.56	2.39	2.57	3.33
25-year	NOAA Data			Precipitation Data			
	Urb Pen	Urb Berry	Foothills	San Jose	Pen WTP	Curtner R	Haskins R
1-Hour	0.67	0.77	0.88	0.85	0.90	0.98	1.00
6-Hour	1.79	2.05	2.35	1.86	1.97	2.07	2.42
12-Hour	2.88	3.30	3.77	2.41	2.51	2.57	3.20
24-Hour	4.15	4.76	5.44	3.07	2.87	3.09	4.00
50-year	NOAA Data			Precipitation Data			
	Urb Pen	Urb Berry	Foothills	San Jose	Pen WTP	Curtner R	Haskins R
1-Hour	0.73	0.85	0.94	0.96	1.01	1.10	1.12
6-Hour	1.95	2.26	2.51	2.09	2.21	2.32	2.72
12-Hour	3.14	3.64	4.04	2.71	2.82	2.88	3.58
24-Hour	4.53	5.25	5.83	3.45	3.22	3.47	4.48
100-year	NOAA Data			Precipitation Data			
	Urb Pen	Urb Berry	Foothills	San Jose	Pen WTP	Curtner R	Haskins R
1-Hour	0.85	0.98	1.12	1.06	1.12	1.22	1.24
6-Hour	2.26	2.60	2.97	2.31	2.44	2.57	3.00
12-Hour	3.63	4.18	4.78	3.00	3.12	3.19	3.96
24-Hour	5.23	6.03	6.89	3.81	3.56	3.84	4.96
200-year	NOAA Data			Precipitation Data			
	Urb Pen	Urb Berry	Foothills	San Jose	Pen WTP	Curtner R	Haskins R
1-Hour	0.93	1.07	1.22	1.16	1.23	1.34	1.36
6-Hour	2.47	2.85	3.24	2.53	2.67	2.81	3.29
12-Hour	3.97	4.59	5.21	3.28	3.41	3.49	4.34
24-Hour	5.72	6.62	7.52	4.17	3.89	4.20	5.42
500-year	NOAA Data			Precipitation Data			
	Urb Pen	Urb Berry	Foothills	San Jose	Pen WTP	Curtner R	Haskins R
1-Hour	1.05	1.21	1.38	1.31	1.36	1.48	1.50
6-Hour	2.78	3.23	3.67	2.86	2.96	3.11	3.64
12-Hour	4.48	5.19	5.89	3.71	3.78	3.87	4.80
24-Hour	6.46	7.48	8.51	4.73	4.31	4.65	6.01

Table 6. 24-Hour Storm Loss Rate Parameters

Subbasin	2-25 Year Basin Model		50-100 Year Basin Model		200-500 Year Basin Model	
	Initial Loss	Constant Loss	Initial Loss	Constant Loss	Initial Loss	Constant Loss
B12	0.23	0.17	0.18	0.14	0.09	0.07
B14	0.23	0.17	0.18	0.14	0.09	0.07
B16	0.23	0.17	0.18	0.14	0.09	0.07
B18	0.23	0.17	0.18	0.14	0.09	0.07
B20	0.23	0.17	0.18	0.14	0.09	0.07
CAL4	0.23	0.17	0.18	0.14	0.09	0.07
CAL6	0.23	0.17	0.18	0.14	0.09	0.07
CAL8	0.23	0.17	0.18	0.14	0.09	0.07
CR4	0.23	0.17	0.18	0.14	0.09	0.07
F2	0.23	0.17	0.18	0.14	0.09	0.07
LC8	0.23	0.17	0.18	0.14	0.09	0.07
LP2	0.23	0.17	0.18	0.14	0.09	0.07
LP4	0.23	0.17	0.18	0.14	0.09	0.07
LP6	0.23	0.17	0.18	0.14	0.09	0.07
LP8	0.23	0.17	0.18	0.14	0.09	0.07
LP10	0.23	0.17	0.18	0.14	0.09	0.07
LP12	0.23	0.17	0.18	0.14	0.09	0.07
LP14	0.23	0.17	0.18	0.14	0.09	0.07
LP16	0.23	0.17	0.18	0.14	0.09	0.07
W4	0.23	0.17	0.18	0.14	0.09	0.07
P4	0.32	0.25	0.27	0.21	0.18	0.14
SIE4	0.32	0.25	0.27	0.21	0.18	0.14
SW4	0.32	0.25	0.27	0.21	0.18	0.14
T4	0.32	0.25	0.27	0.21	0.18	0.14
B2	0.54	0.35	0.45	0.28	0.24	0.21
B4	0.54	0.35	0.45	0.28	0.24	0.21
B6	0.54	0.35	0.45	0.28	0.24	0.21
B8	0.54	0.35	0.45	0.28	0.24	0.21
B10	0.54	0.35	0.45	0.28	0.24	0.21
CAL2	0.54	0.35	0.45	0.28	0.24	0.21
CR2	0.54	0.35	0.45	0.28	0.24	0.21
LC2	0.54	0.35	0.45	0.28	0.24	0.21
LC4	0.54	0.35	0.45	0.28	0.24	0.21
LC6	0.54	0.35	0.45	0.28	0.24	0.21
P2	0.54	0.35	0.45	0.28	0.24	0.21
SIE2	0.54	0.35	0.45	0.28	0.24	0.21
ST2	0.54	0.35	0.45	0.28	0.24	0.21
SW2	0.54	0.35	0.45	0.28	0.24	0.21
T2	0.54	0.35	0.45	0.28	0.24	0.21

Table 7. Baseflow Parameters

Areas	Initial Flow (cfs/mi^2)	Recession Ratio	Threshold Flow (Peak Ratio)
Rural Hillsides	4	0.25	0.15
Urban to Rural Transition	1	0.10	0.10
Pervious Urban Floodplain	1	0.10	0.10

Table 8. Percent Impervious by Land-use Category

Category	% Impervious	% Pervious
Commercial Industrial (CI)	85	15
High Density Residential (R-1)	48	52
Medium Density Residential (R-2)	30	76
Low Density Residential (R-3)	12	88
Rural Residential (R-4)	4	96
Park (Pk)	5	95
Grassland (GL)	0	100
Forested (F)	0	100

Table 9. Percent Impervious and Pervious by Subbasin

	Existing Condition		Future Condition		
Subbasin	% Impervious	% Pervious	% Impervious	% Pervious	Area (mi^2)
B2	0.5	99.5	4.0	96.0	1.03
B4	0.0	100.0	4.0	96.0	0.37
B6	0.0	100.0	4.0	96.0	0.81
B8	0.2	99.8	4.0	96.0	1.20
B10	0.5	99.5	4.0	96.0	0.99
B12	42.0	58.0	43.0	57.0	1.21
B14	76.0	24.0	85.0	15.0	0.29
B16	68.0	32.0	81.0	19.0	0.10
B18	57.0	43.0	70.0	30.0	0.21
B20	40.0	60.0	42.0	58.0	0.54
CAL2	2.0	98.0	6.0	94.0	2.49
CAL4	36.0	64.0	52.0	48.0	0.39
CAL6	76.0	24.0	78.0	22.0	0.06
CAL8	37.0	63.0	45.0	55.0	0.65
CR2	1.0	99.0	5.0	95.0	0.51
CR4	37.0	63.0	37.0	63.0	0.33
F2	70.0	30.0	82.0	18.0	0.56
LC2	3.0	97.0	6.0	94.0	1.57
LC4	2.0	98.0	6.0	94.0	1.40

Table 9. Percent Impervious and Pervious by Subbasin (Cont.)

Subbasin	Existing Condition		Future Condition		Area (mi^2)
	% Impervious	% Pervious	% Impervious	% Pervious	
LC6	4.0	96.0	7.0	93.0	0.41
LC8	49.0	51.0	53.0	47.0	0.40
LP2	29.0	71.0	58.0	42.0	2.98
LP4	56.0	44.0	56.0	44.0	1.31
LP6	39.0	61.0	41.0	59.0	0.19
LP8	36.0	64.0	36.0	64.0	0.11
LP10	38.0	62.0	44.0	56.0	0.26
LP12	69.0	31.0	85.0	15.0	0.14
LP14	60.0	40.0	71.0	29.0	0.15
LP16	79.0	21.0	79.0	21.0	0.09
P2	0.0	100.0	4.0	96.0	0.50
P4	34.0	66.0	39.0	61.0	1.53
SIE2	6.0	94.0	10.0	90.0	0.42
SIE4	36.0	64.0	36.0	64.0	1.53
ST2	15.0	85.0	15.0	85.0	0.38
SW2	3.0	97.0	6.0	94.0	0.25
SW4	20.0	80.0	22.0	78.0	0.25
T2	6.0	94.0	8.0	92.0	0.41
T4	29.0	71.0	35.0	65.0	1.37
W2	70.0	30.0	79.0	21.0	0.85

Table 10. Basin Lag Parameters

Subbasin	Slope (feet/mile)	Longest Flow Length (L) (miles)	Centroidal Length (L_{CA}) (miles)	Basin Ave Roughness (n)	Basin Lag $24n((LL_{CA})/S^{1/2})^{0.38}$ (hours)
B2	301	2.44	0.77	0.10	1.03
B4	465	1.35	0.64	0.10	0.71
B6	612	2.18	0.97	0.10	0.94
B8	465	2.37	0.81	0.10	0.96
B10	396	2.51	1.24	0.10	1.19
CAL2	734	3.40	1.79	0.10	1.36
CR2	676	1.88	1.01	0.10	0.89
LC2	554	2.80	0.68	0.10	0.92
LC4	375	2.25	1.11	0.10	1.10
LC6	449	1.51	0.72	0.10	0.78
P2	697	1.46	0.60	0.10	0.66
SIE2	628	2.41	1.37	0.10	1.11
ST2	486	1.25	0.80	0.10	0.74
SW2	660	1.71	1.03	0.10	0.87
T2	401	1.33	0.67	0.10	0.73

Table 11. Kinematic Wave Parameters

B12	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	42%	60	0.018	0.100				
Overland (P)	58%	200	0.010	0.250				
Sub-collector		400	0.018	0.013	Trapezoid	8	0.009	0
Main		1,500	0.005	0.018	Circular			2

B14	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	76%	140	0.0086	0.100				
Overland (P)	24%	200	0.0125	0.250				
Sub-collector		400	0.0086	0.013	Trapezoid	8	0.008	0
Main		1,500	0.0110	0.018	Circular			2

B16	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	68%	140	0.0060	0.100				
Overland (P)	32%	180	0.0125	0.250				
Main		110	0.0125	0.018	Circular			2

B18	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	57%	70	0.0025	0.100				
Overland (P)	43%	100	0.0031	0.250				
Sub-collector		400	0.0025	0.013	Trapezoid	8	0.007	0
Collector		660	0.0030	0.018	Circular		0.070	2
Main		400	0.0024	0.018	Circular			5

B20	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	40%	60	0.0040	0.100				
Overland (P)	60%	100	0.0056	0.250				
Sub-collector		400	0.0040	0.013	Trapezoid	8	0.007	0
Collector		800	0.0040	0.018	Circular		0.080	2.5
Main		2,500	0.0040	0.018	Circular			5

Table 11. Kinematic Wave Parameters (Cont.)

CAL4	% Total of Subbasin	L (ft)	S (ft/ft)	Roughness	Channel Shape	Z	Collector System Area (mi^2)	Bottom Width or Diam (ft)
Overland (I)	36%	70	0.007	0.100				
Overland (P)	64%	150	0.014	0.250				
Sub-collector		400	0.007	0.013	Trapezoid	8	0.009	0
Collector		2,180	0.003	0.018	Circular		0.130	2
Main		6,240	0.011	0.070	Trapezoid	1		17
CAL6	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	76%	120	0.005	0.100				
Overland (P)	24%	120	0.010	0.250				
Sub-collector		400	0.005	0.013	Trapezoid	8	0.007	0
Collector		200	0.005	0.018	Circular		0.060	1
Main		900	0.005	0.018	Circular			1.125
CAL8	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	37%	60	0.015	0.100				
Overland (P)	63%	150	0.020	0.250				
Sub-collector		400	0.015	0.013	Trapezoid	8	0.010	0
Collector		800	0.015	0.018	Circular		0.093	3
Main		500	0.008	0.018	Circular			6
CR4	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	37%	60	0.0220	0.100				
Overland (P)	63%	200	0.0250	0.250				
Sub-collector		400	0.0220	0.013	Trapezoid	8	0.007	0
Collector		1,500	0.0167	0.018	Circular		0.066	2
Main		4,920	0.0223	0.027	Trapezoid	1		15
F2	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	70%	120	0.0040	0.100				
Overland (P)	30%	200	0.0036	0.250				
Sub-collector		400	0.0035	0.018	Circular		0.009	2
Collector		820	0.0035	0.018	Circular		0.112	2.5
Main		6,240	0.0020	0.080	Trapezoid	1		12

Table 11. Kinematic Wave Parameters (Cont.)

LC8	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	49%	60	0.0125	0.100				
Overland (P)	51%	200	0.0160	0.250				
Sub-collector		400	0.0125	0.013	Trapezoid	8	0.007	0
Collector		1,500	0.0050	0.018	Circular		0.068	2
Main		5,470	0.0237	0.020	Trapezoid	1		4
LP2	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	29%	70	0.0058	0.100				
Overland (P)	71%	200	0.0060	0.250				
Sub-collector		400	0.0058	0.013	Trapezoid	8	0.008	0
Collector		1,320	0.0050	0.018	Circular		0.099	2
Main		10,560	0.0020	0.050	Trapezoid	1		20
LP4	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	56%	80	0.0025	0.100				
Overland (P)	44%	200	0.0025	0.250				
Sub-collector		400	0.0025	0.013	Trapezoid	8	0.007	0
Collector		2,000	0.0050	0.018	Circular		0.082	4
Main		7,320	0.0020	0.050	Trapezoid	1		20
LP6	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	39%	60	0.001	0.100				
Overland (P)	61%	100	0.001	0.250				
Sub-collector		400	0.001	0.013	Trapezoid	8	0.008	0
Collector		900	0.001	0.018	Circular		0.095	2.5
Main		200	0.001	0.018	Circular			4
LP8	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	34%	60	0.001	0.100				
Overland (P)	66%	90	0.001	0.250				
Sub-collector		400	0.001	0.013	Trapezoid	8	0.005	0
Collector		800	0.001	0.018	Circular		0.037	2
Main		400	0.001	0.018	Circular			2.5

Table 11. Kinematic Wave Parameters (Cont.)

LP10	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	38%	65	0.001	0.100				
Overland (P)	62%	100	0.001	0.250				
Sub-collector		400	0.001	0.013	Trapezoid	8	0.007	0
Collector		800	0.001	0.018	Circular		0.065	2
Main		800	0.001	0.018	Circular			4.5

LP12	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	69%	120	0.001	0.100				
Overland (P)	31%	150	0.001	0.250				
Sub-collector		400	0.001	0.013	Trapezoid	8	0.005	0
Main		2,160	0.001	0.018	Circular			3

LP14	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	40%	70	0.001	0.100				
Overland (P)	60%	50	0.001	0.250				
Sub-collector		400	0.001	0.013	Circular		0.004	1.5
Collector		800	0.001	0.018	Circular		0.025	2.5
Main		700	0.001	0.018	Circular			4

LP16	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	21%	50	0.001	0.100				
Overland (P)	79%	50	0.001	0.250				
Sub-collector		400	0.001	0.013	Trapezoid	8	0.004	0
Main		700	0.001	0.018	Circular			2.5

P4	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	34%	60	0.007	0.100				
Overland (P)	66%	200	0.020	0.250				
Sub-collector		400	0.007	0.013	Trapezoid	8	0.008	0
Collector		1,500	0.008	0.018	Circular		0.109	2
Main		7,540	0.018	0.020	Trapezoid	Wall		8

Table 11. Kinematic Wave Parameters (Cont.)

SIE4	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	36%	60	0.0180	0.100				
Overland (P)	64%	200	0.0500	0.250				
Sub-collector		400	0.0180	0.013	Trapezoid	8	0.009	0
Collector		1,500	0.0077	0.018	Circular		0.102	2
Main		9,300	0.0110	0.030	Trapezoid	2		5

SW4	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	20%	70	0.0870	0.100				
Overland (P)	80%	200	0.1000	0.250				
Sub-collector		400	0.0870	0.013	Trapezoid	8	0.008	0
Collector		1,100	0.0670	0.018	Circular		0.083	2
Main		1,440	0.0556	0.050	Trapezoid	0.5		6

T4	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	29%	60	0.0120	0.100				
Overland (P)	71%	200	0.0200	0.250				
Sub-collector		400	0.0120	0.013	Trapezoid	8	0.010	0
Collector		1,500	0.0088	0.018	Circular		0.137	2
Main		5,250	0.0200	0.065	Trapezoid	1		10

W2	**% Total of Subbasin**	**L (ft)**	**S (ft/ft)**	**Roughness**	**Channel Shape**	**Z**	**Collector System Area (mi^2)**	**Bottom Width or Diam (ft)**
Overland (I)	70%	100	0.0045	0.100				
Overland (P)	30%	200	0.0041	0.250				
Sub-collector		400	0.0045	0.013	Trapezoid	8	0.008	0
Collector		1,500	0.0040	0.018	Circular		0.085	2.5
Main		9,630	0.0036	0.080	Trapezoid	1		12

Table 12. Stream Reach Characteristics and Muskingum Routing Parameters

Reach Name	**Reach Length (miles)**	**Reach Slope (ft/ft)**	**Velocity (ft/s)**	**Muskingum K (hours)**	**Musk. X**	**N steps**
RC1-LC2	1.204	0.0408	6.0	0.36	0.25	3
RB1-B3	1.473	0.0831	6.0	0.46	0.25	3
RB3-B5	1.872	0.0544	6.0	0.29	0.25	2

Table 13. Modified Puls Parameters

Node From B33 To B31	Volume (acre-ft)	Discharge (cfs)
	9.41	830
	9.50	1,660
	9.69	2,650
	10.75	5,340
	15.92	10,200
	17.82	12,200
	19.40	14,200

Node From B31 To B29	Volume (acre-ft)	Discharge (cfs)
	45.90	830
	51.20	1,660
	59.12	2,650
	82.28	5,340
	136.99	10,200
	159.82	12,200
	199.42	14,200

Node From B29 To B27	Volume (acre-ft)	Discharge (cfs)
	6.76	830
	9.68	1,660
	12.69	2,650
	19.58	5,340
	30.23	10,200
	37.80	12,200
	47.79	14,200

Node From B27 To B25	Volume (acre-ft)	Discharge (cfs)
	12.23	830
	18.41	1,660
	24.51	2,650
	38.12	5,340
	57.90	10,200
	69.49	12,200
	85.04	14,200

Node From B25 To B23	Volume (acre-ft)	Discharge (cfs)
	12.54	830
	19.50	1,660
	30.81	2,650
	61.33	5,340
	88.25	10,200
	97.72	12,200
	107.28	14,200

Node From B23 To B21	Volume (acre-ft)	Discharge (cfs)
	23.25	830
	38.16	1,660
	59.90	2,650
	99.00	4,000
	130.82	5,340
	198.76	10,200
	221.76	12,200
	244.60	14,200

Table 13. Modified Puls Parameters (Cont.)

Node From B21 To B19	Volume (acre-ft)	Discharge (cfs)
	6.12	830
	9.73	1,660
	13.61	2,650
	29.96	5,340
	54.63	10,200
	62.66	12,200
	70.59	14,200

Node From B19 To B17(a)	Volume (acre-ft)	Discharge (cfs)
	0.52	214
	0.89	428
	1.50	857
	2.04	1,285
	2.54	1,714
	3.00	2,142
	4.91	3,296
	6.23	4,072
	7.14	4,759
	9.44	6,735
	10.80	8,000

Node From B17(a) To B17	Volume (acre-ft)	Discharge (cfs)
	1.43	162
	2.33	324
	3.81	648
	5.18	973
	6.46	1,297
	7.62	1,621
	10.25	2,388
	11.99	2,943
	13.61	3,446
	18.22	5,094
	21.00	6,000

Node From B17 To B15	Volume (acre-ft)	Discharge (cfs)
	0.96	143
	1.61	285
	2.74	571
	3.71	856
	4.56	1,141
	5.33	1,426
	7.09	2,089
	8.40	2,551
	9.57	2,968
	13.33	4,230
	15.50	5,000

Node From B15 To B13	Volume (acre-ft)	Discharge (cfs)
	2.88	130
	4.73	259
	7.79	518
	10.43	777
	12.76	1,037
	14.97	1,296
	19.80	1,895
	22.93	2,304
	25.57	2,674
	34.01	3,741
	40.00	4,500

Node From B13 To B11	Volume (acre-ft)	Discharge (cfs)
	4.94	130
	7.91	259
	12.94	518
	17.39	777
	21.55	1,037
	25.56	1,296
	34.30	1,895
	39.60	2,304
	44.62	2,674
	58.31	3,741
	70.00	4,500

Table 13. Modified Puls Parameters (Cont.)

	Node	Volume (acre-ft)	Discharge (cfs)
From	B11		
To	B9		
		1.19	90
		1.88	181
		3.38	361
		4.98	542
		6.24	722
		7.40	903
		12.05	1,392
		14.32	1,742
		16.04	2,054
		22.17	2,854
		25.00	3,250

	Node	Volume (acre-ft)	Discharge (cfs)
From	B9		
To	B7		
		0.77	66
		1.27	131
		2.47	263
		3.64	394
		5.56	526
		6.31	657
		8.84	1,055
		10.56	1,350
		11.95	1,604
		15.99	2,218
		20.00	3,000

	Node	Volume (acre-ft)	Discharge (cfs)
From	B7		
To	B5		
		0.50	66
		0.90	131
		1.47	263
		1.89	394
		2.27	526
		2.59	657
		3.46	1,055
		4.02	1,350
		4.56	1,604
		5.94	2,218
		7.20	3,000

Table 14. Pump Station Data

Abbott		
Elevation (ft)	**Area (ac)**	**Outflow (cfs)**
0.0	0.00	0.0
3.0	0.01	0.0
8.0	3.11	0.0
8.5	3.43	11.9
9.5	4.05	23.8
10.0	4.36	23.8
12.0	6.42	23.8

Berryessa		
Elevation (ft)	**Area (ac)**	**Outflow (cfs)**
0.0	0.00	0
2.0	2.47	0
4.5	3.09	0
6.0	3.46	50
6.5	3.58	100
7.0	3.70	150
10.0	4.44	150
12.0	4.93	150
20.0	30.0	150

Table 14. Pump Station Data (Cont.)

Cal Circle		
Elevation (ft)	**Area (ac)**	**Outflow (cfs)**
0.0	0.00	0
2.7	0.01	0
4.0	1.06	0
4.5	1.11	0
5.8	1.25	38
6.0	1.27	38
7.5	1.42	76
8.0	1.47	76
9.3	1.61	114
10.0	1.68	114
12.0	2.62	114
14.0	3.20	114

Penitencia		
Elevation (ft)	**Area (ac)**	**Outflow (cfs)**
0.0	0.00	0
5.3	0.01	0
6.3	0.01	20
7.0	0.01	20
8.0	2.18	20
8.3	2.22	40
9.8	2.44	60
12.0	2.75	60
14.0	6.00	60

Jurgens	
Storage (ac-ft)	**Outflow (cfs)**
0.00	0
1.03	150
50.00	150

Manor	
Storage (ac-ft)	**Outflow (cfs)**
0.00	0
0.41	60
30.0	60

Minnis	
Storage (ac-ft)	**Outflow (cfs)**
0.00	0
0.21	30
50.00	30

Spence Creek	
Storage (ac-ft)	**Outflow (cfs)**
0.00	0
0.62	90
50.00	90

Wrigley-Ford	
Storage (ac-ft)	**Outflow (cfs)**
0.00	0
2.98	432
100.00	432
150.00	432
200.00	432

Table 15. 24-Hour Storm Flood Frequency Quantiles

Location	Drainage Area (sq mi)	HMS Node	Recurrence Interval (yrs)	Existing Conditions (cfs)	Future Conditions (cfs)
Lower Penitencia Creek Above Confluence with Coyote Creek	29.1	B33	2	2,150	2,520
			5	3,370	3,660
			10	4,030	4,340
			25	5,780	5,940
			50	6,750	6,930
			100	8,100	8,170
			200	9,790	9,860
			500	11,030	11,060
Lower Penitencia Creek Below Confluence with Berryessa Creek	28.2	B29	2	2,040	2,330
			5	3,210	3,540
			10	3,890	4,210
			25	5,660	5,780
			50	6,600	6,740
			100	7,890	7,960
			200	9,630	9,690
			500	11,030	11,060
Berryessa Creek at Downstream End Immediately Above Confluence w/ Lower Penitencia Creek	22.4	B27	2	1,620	1,730
			5	2,580	2,710
			10	3,050	3,180
			25	4,410	4,510
			50	5,270	5,390
			100	6,430	6,480
			200	8,010	8,070
			500	9,180	9,210
Berryessa Creek at North Abel Street Below Confluence with Wrigley-Ford Ditch	19.4	B25	2	1,500	1,620
			5	2,320	2,420
			10	2,730	2,850
			25	3,900	3,980
			50	4,620	4,710
			100	5,580	5,610
			200	6,870	6,910
			500	8,140	8,190
Berryessa Creek Berryessa Pump Station	17.8	B23	2	1,300	1,370
			5	2,040	2,140
			10	2,460	2,550
			25	3,540	3,610
			50	4,280	4,380
			100	5,250	5,280
			200	6,470	6,490
			500	7,600	7,630
Berryessa Creek Below Confluence with Tularcitos Creek	17.3	B21	2	1,210	1,280
			5	1,940	2,020
			10	2,330	2,410
			25	3,490	3,550
			50	4,180	4,280
			100	5,250	5,260
			200	6,190	6,230
			500	7,250	7,310

Table 15. 24-Hour Storm Flood Frequency Quantiles (Cont.)

Location	**Drainage Area** (sq mi)	**HMS Node**	**Recurrence Interval** (yrs)	**Existing Conditions** (cfs)	**Future Conditions** (cfs)
Berryessa Creek at Calaveras Boulevard	15.10	B19	2	1,030	1,090
			5	1,670	1,730
			10	1,990	2,050
			25	2,950	3,040
			50	3,650	3,740
			100	4,690	4,700
			200	5,440	5,490
			500	6,430	6,480
Berryessa Creek Below Confluence with Piedmont Creek	11.2	B17	2	780	840
			5	1,290	1,350
			10	1,550	1,600
			25	2,400	2,450
			50	2,930	2,990
			100	3,780	3,800
			200	4,490	4,520
			500	5,180	5,230
Berryessa Creek at Yosemite Rd	9.20	B15	2	580	620
			5	960	990
			10	1,130	1,170
			25	1,730	1,770
			50	2,150	2,200
			100	2,890	2,910
			200	3,550	3,580
			500	4,240	4,290
Berryessa Creek at Landess Road	8.91	B13	2	590	610
			5	940	960
			10	1,080	1,120
			25	1,580	1,620
			50	1,970	2,020
			100	2,770	2,780
			200	3,450	3,490
			500	4,150	4,200
Berryessa Creek Below Confluence with Sierra Creek (Morrill Rd)	7.70	B11	2	440	470
			5	680	710
			10	790	830
			25	1,210	1,260
			50	1,600	1,630
			100	2,100	2,140
			200	2,630	2,660
			500	3,090	3,140
Berryessa Creek Below Confluence with Crosley Creek	5.75	B9	2	280	300
			5	480	500
			10	650	700
			25	970	1,000
			50	1,300	1,340
			100	1,700	1,740
			200	2,190	2,220
			500	2,580	2,600

Table 15. 24-Hour Storm Flood Frequency Quantiles (Cont.)

Location	Drainage Area (sq mi)	HMS Node	Recurrence Interval (yrs)	Existing Conditions (cfs)	Future Conditions (cfs)
Berryessa Creek Below Confluence with Sweigert Creek	4.91	B7	2	230	260
			5	420	450
			10	570	600
			25	850	890
			50	1,150	1,180
			100	1,500	1,530
			200	1,940	1,960
			500	2,270	2,300
Berryessa Creek at Old Piedmont Road	4.40	B5	2	220	240
			5	390	420
			10	530	560
			25	790	830
			50	1,060	1,090
			100	1,400	1,430
			200	1,800	1,820
			500	2,110	2,130

Table 16. Regression Equation Analysis

Basin ID	Slope (ft/ft)	Drainage Area (mi^2)	Basin Length (mi)	Mean Annual Precip (MAP) (in)	100-Year Peak Flow (Q_{100}) (cfs)	90% Confidence		Existing Q_{100} (from HMS) (cfs)
						Lower Limit (cfs)	Upper Limit (cfs)	
B5	0.0754	4.40	5.02	19	608	277	1,335	1,398
B2	0.0667	1.03	2.04	21	273	109	681	352
CAL2	0.1302	2.49	2.35	19	357	136	784	728

Table 17. Flood Frequency Analysis At Two Gage Stations

Gage	10-Year Q_{10} (cfs)	100-Year Q_{100} (cfs)	Area (mi^2)	Q_{100} / Area (cfs/mi^2)
SCVWD Station 64	2,260	4,610	15.1	305
USGS Station 11172100	1,811	4,416	21.5	205

Table 18. Comparison of Peak Flows with Other Berryessa Creek Studies

Location	10-year Recurrence			100-year Recurrence		
	COE 1982 (cfs)	SCVWD 1978 (cfs)	nhc 2003 (cfs)	COE 1982 (cfs)	SCVWD 1978 (cfs)	nhc 2003 (cfs)
at Old Piedmont Road	640	700	530	1,570	1,580	1,400
Below Piedmont Creek	1,610	1,720	1,550	3,420	2,980	3,780
at Calaveras Boulevard	2,150	2,270	1,990	4,780	4,020	4,690

Table 19. 72-Hour Rainfall Distribution

Time (hrs)	% of Storm Total	Time (hrs)	% of Storm Total	Time (hrs)	% of Storm Total
1	0.00	25	3.12	49	0.00
2	0.85	26	4.63	50	0.00
3	1.76	27	3.36	51	0.78
4	3.88	28	1.34	52	2.54
5	3.50	29	1.35	53	1.96
6	3.88	30	1.14	54	2.35
7	4.10	31	0.39	55	2.54
8	3.88	32	0.14	56	2.26
9	1.76	33	0.42	57	1.05
10	1.06	34	1.31	58	0.39
11	0.99	35	1.88	59	0.31
12	0.49	36	1.54	60	0.08
13	0.14	37	1.36	61	0.00
14	0.35	38	2.15	62	0.00
15	0.00	39	5.14	63	0.00
16	0.00	40	5.68	64	0.00
17	0.00	41	5.95	65	0.00
18	0.00	42	3.64	66	0.00
19	0.00	43	2.63	67	0.00
20	0.00	44	2.32	68	0.00
21	0.07	45	3.12	69	0.04
22	0.21	46	3.26	70	0.12
23	0.07	47	1.72	71	0.04
24	0.00	48	0.94	72	0.00

Table 20. 72-Hour Precipitation Totals

Region	72-Hour Precipitation Depth (inches)							
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr	500-yr
Upper Penitencia	3.50	4.78	5.42	7.01	7.66	8.84	9.67	10.92
Upper Berryessa	3.94	5.42	6.08	8.04	8.87	10.19	11.19	12.64
Foothills	4.43	6.08	7.23	9.19	9.85	11.64	12.71	14.38

Table 21. 72-Hour Storm Loss Rate Parameters

Subbasin	2-25 Yr Basin Model		50-100 Yr Basin Model		200-500 Yr Basin Model	
	Initial Loss	Constant Loss	Initial Loss	Constant Loss	Initial Loss	Constant Loss
B12	0.23	0.106	0.18	0.090	0.09	0.045
B14	0.23	0.106	0.18	0.090	0.09	0.045
B16	0.23	0.106	0.18	0.090	0.09	0.045
B18	0.23	0.106	0.18	0.090	0.09	0.045
B20	0.23	0.106	0.18	0.090	0.09	0.045
CAL4	0.23	0.106	0.18	0.090	0.09	0.045
CAL6	0.23	0.106	0.18	0.090	0.09	0.045
CAL8	0.23	0.106	0.18	0.090	0.09	0.045
CR4	0.23	0.106	0.18	0.090	0.09	0.045
F2	0.23	0.106	0.18	0.090	0.09	0.045
LC8	0.23	0.106	0.18	0.090	0.09	0.045
LP2	0.23	0.106	0.18	0.090	0.09	0.045
LP4	0.23	0.106	0.18	0.090	0.09	0.045
LP6	0.23	0.106	0.18	0.090	0.09	0.045
LP8	0.23	0.106	0.18	0.090	0.09	0.045
LP10	0.23	0.106	0.18	0.090	0.09	0.045
LP12	0.23	0.106	0.18	0.090	0.09	0.045
LP14	0.23	0.106	0.18	0.090	0.09	0.045
LP16	0.23	0.106	0.18	0.090	0.09	0.045
W4	0.23	0.106	0.18	0.090	0.09	0.045
P4	0.32	0.161	0.27	0.135	0.18	0.090
SIE4	0.32	0.161	0.27	0.135	0.18	0.090
SW4	0.32	0.161	0.27	0.135	0.18	0.090
T4	0.32	0.161	0.27	0.135	0.18	0.090
B2	0.54	0.225	0.45	0.180	0.24	0.135
B4	0.54	0.225	0.45	0.180	0.24	0.135
B6	0.54	0.225	0.45	0.180	0.24	0.135
B8	0.54	0.225	0.45	0.180	0.24	0.135
B10	0.54	0.225	0.45	0.180	0.24	0.135
CAL2	0.54	0.225	0.45	0.180	0.24	0.135
CR2	0.54	0.225	0.45	0.180	0.24	0.135
LC2	0.54	0.225	0.45	0.180	0.24	0.135
LC4	0.54	0.225	0.45	0.180	0.24	0.135
LC6	0.54	0.225	0.45	0.180	0.24	0.135
P2	0.54	0.225	0.45	0.180	0.24	0.135
SIE2	0.54	0.225	0.45	0.180	0.24	0.135
ST2	0.54	0.225	0.45	0.180	0.24	0.135
SW2	0.54	0.225	0.45	0.180	0.24	0.135
T2	0.54	0.225	0.45	0.180	0.24	0.135

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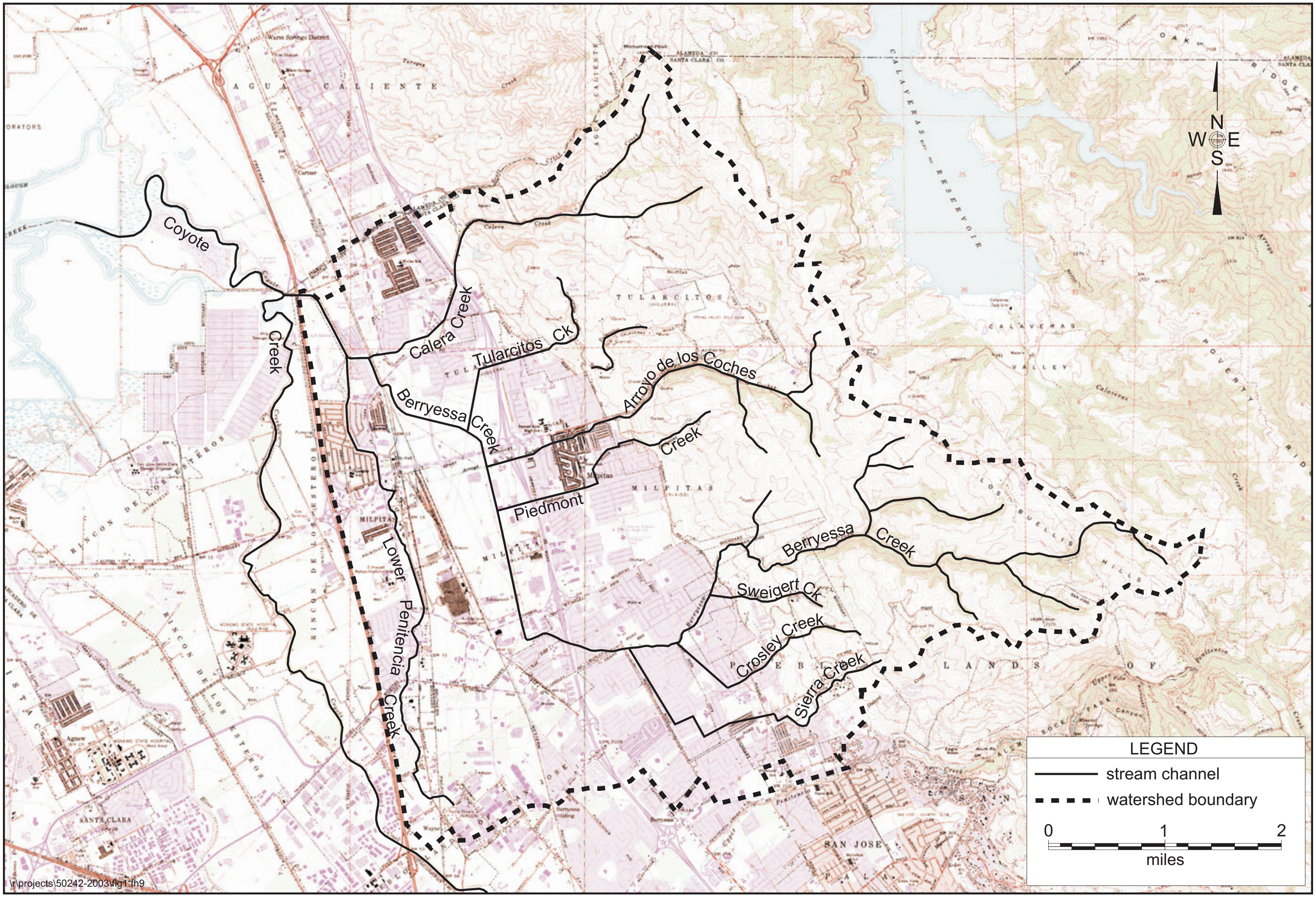

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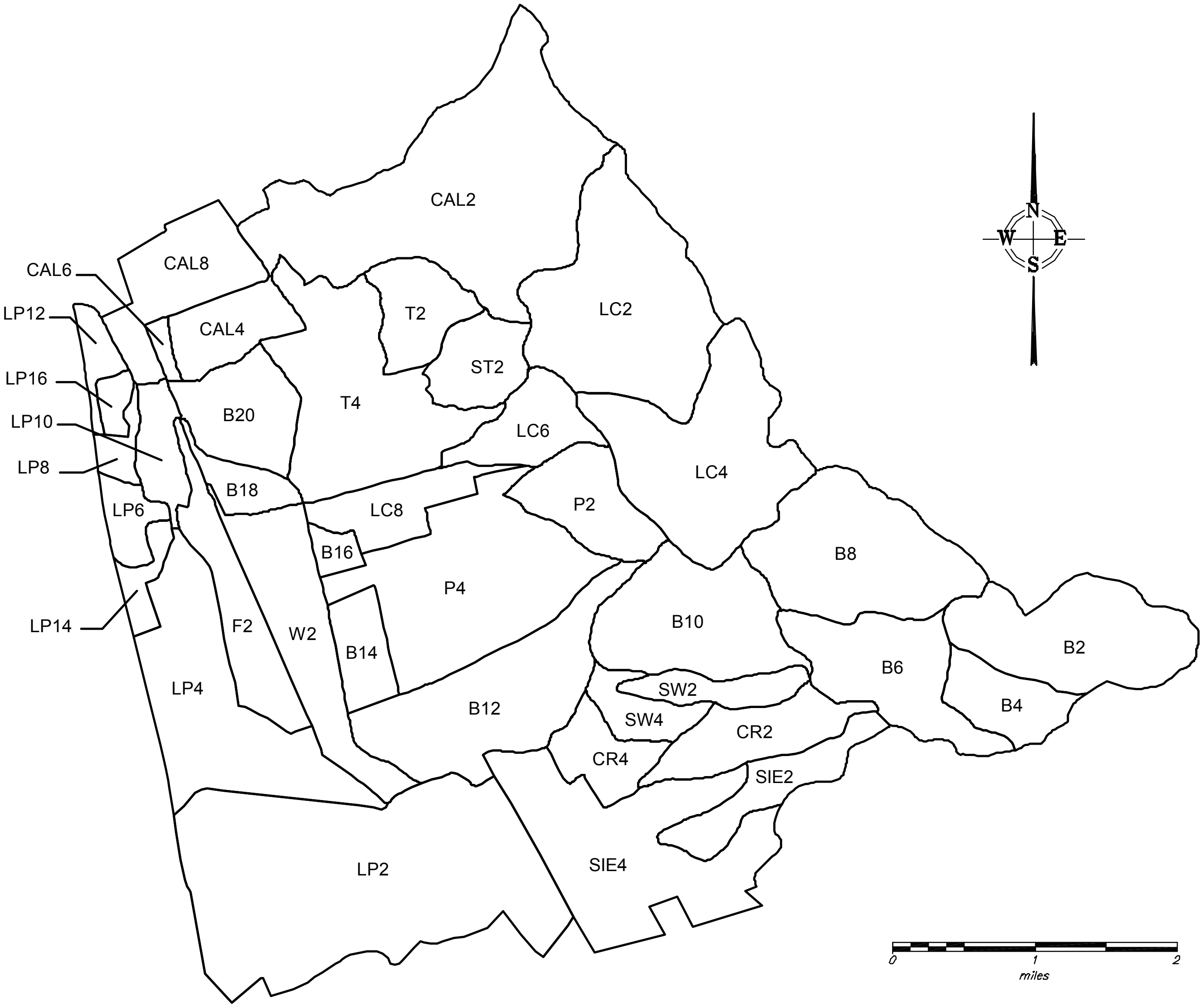
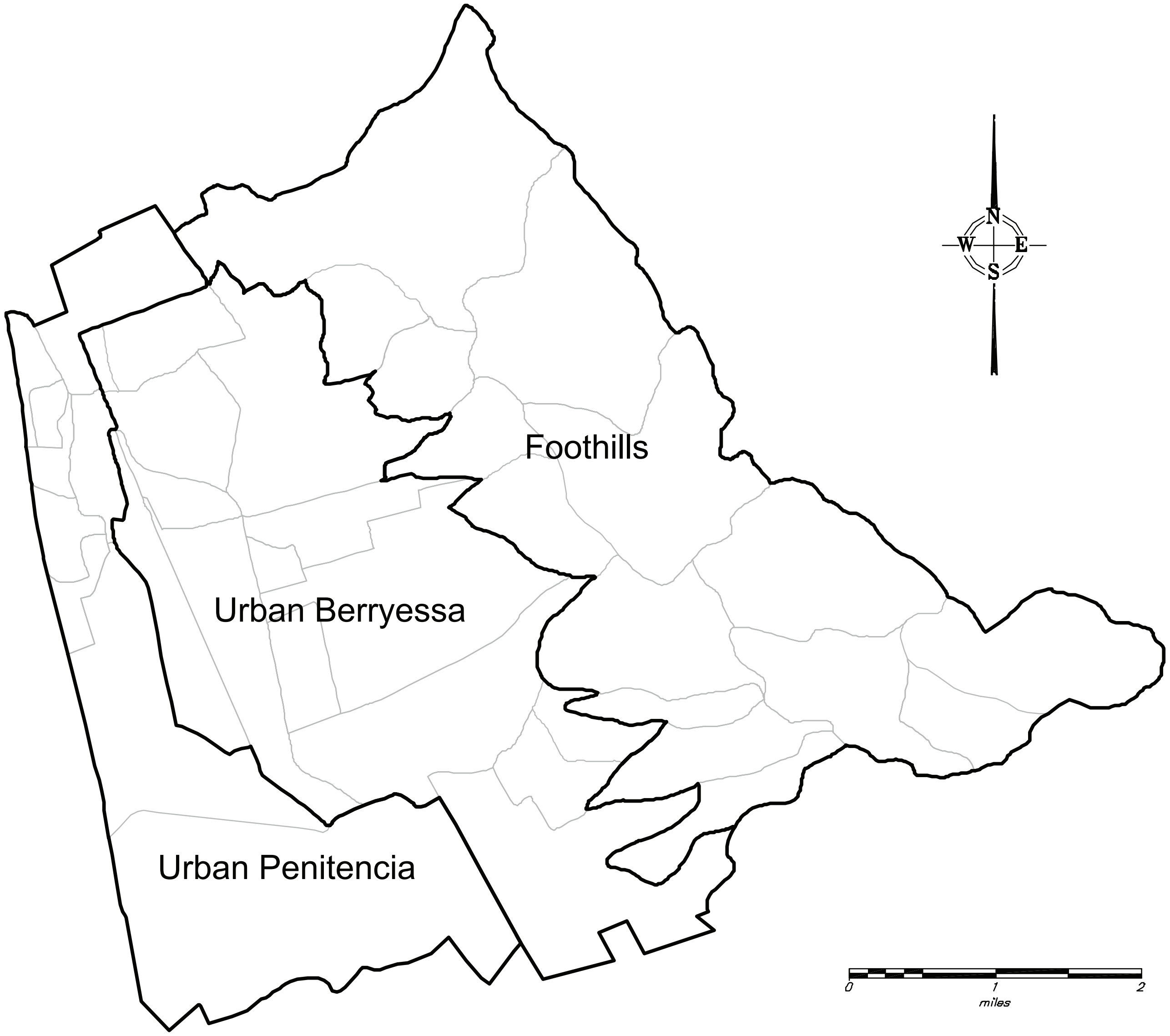
Berryessa Creek Levees Project

Figure 1

Berryessa Creek Watershed Map



HEC-HMS Subbasin Map
Figure 2



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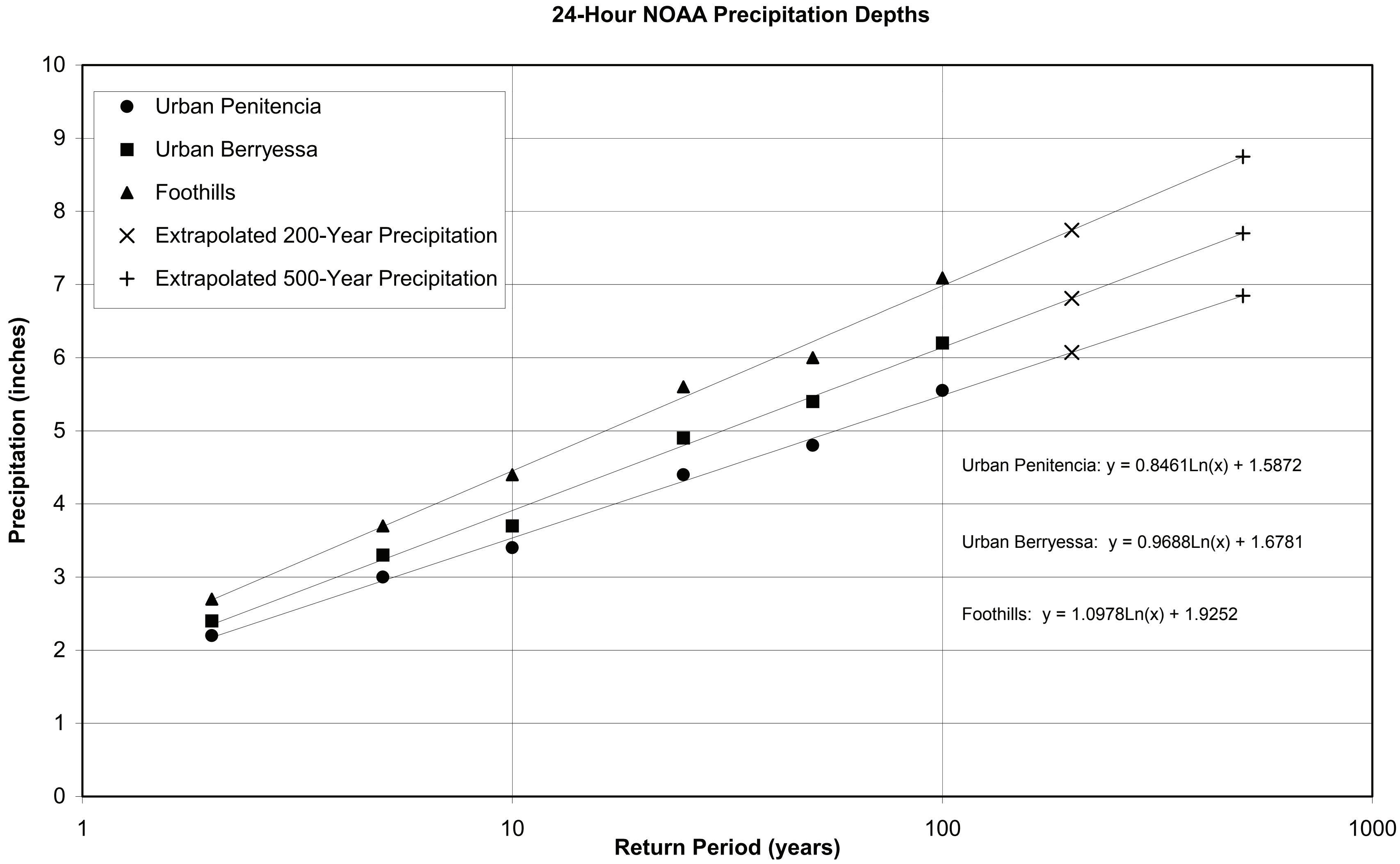
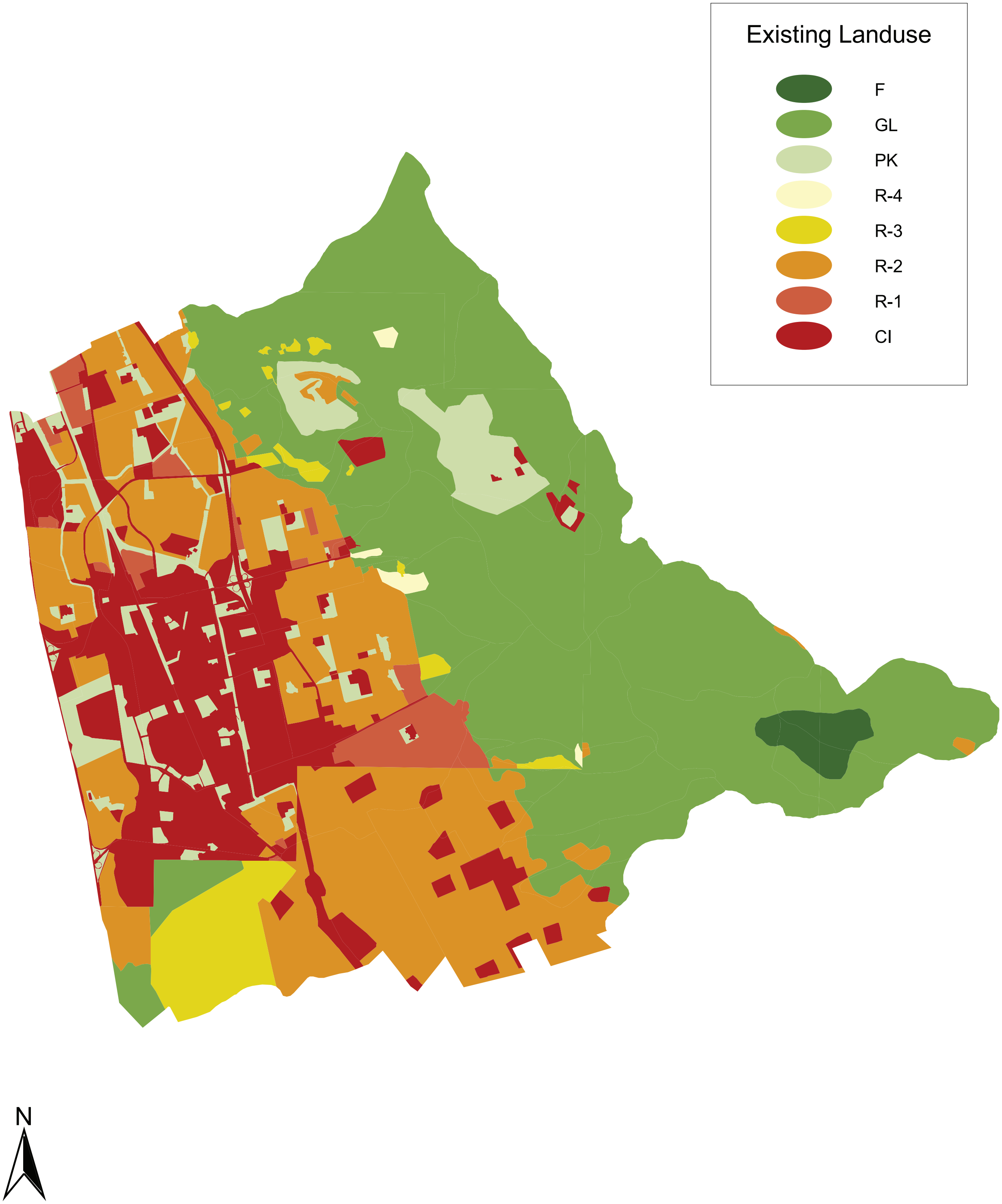
Berryessa Creek Levees Project

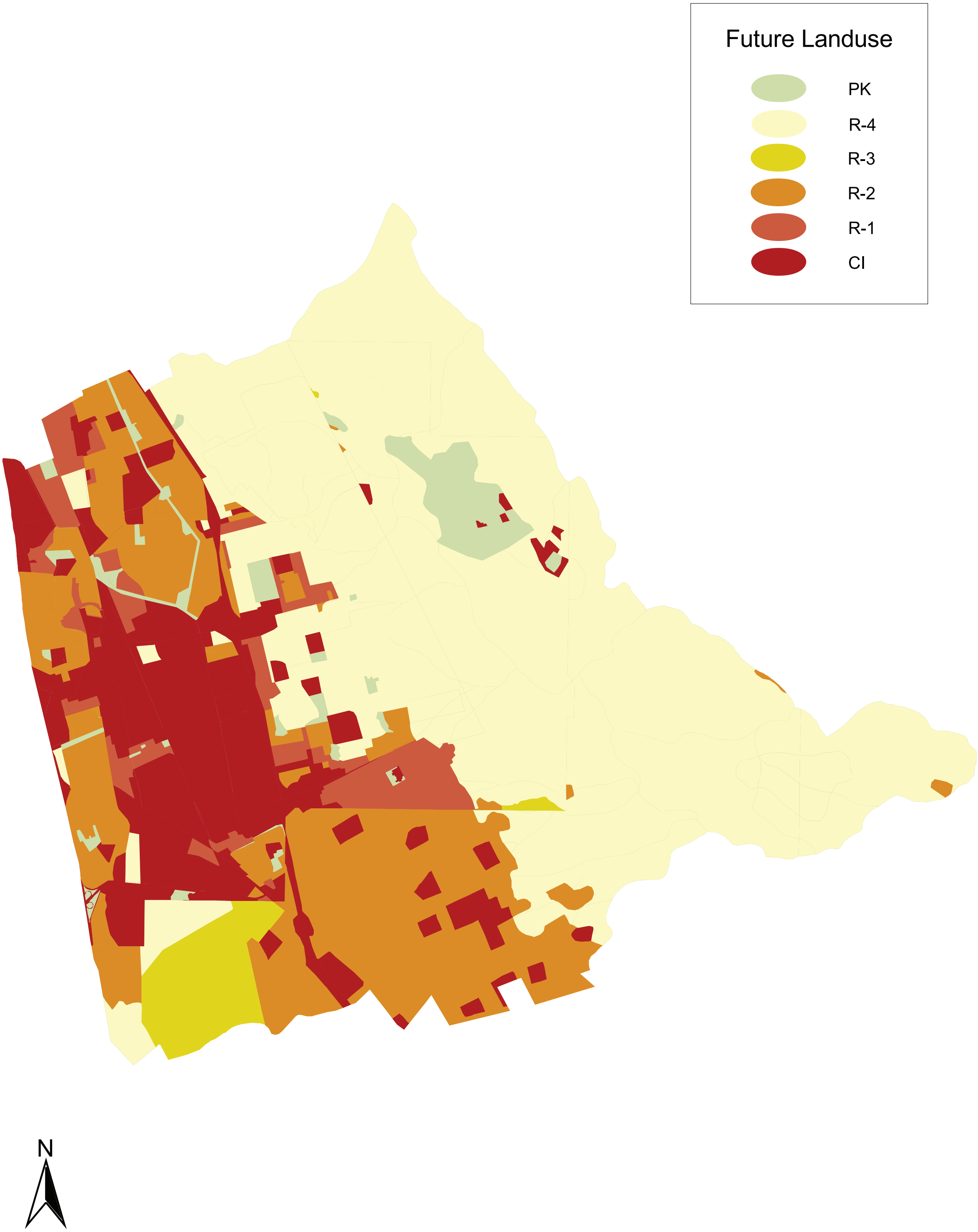
Precipitation Regions
Figure 3



Note: Return Period axis is plotted on a log scale.



Existing Landuse
Figure 5



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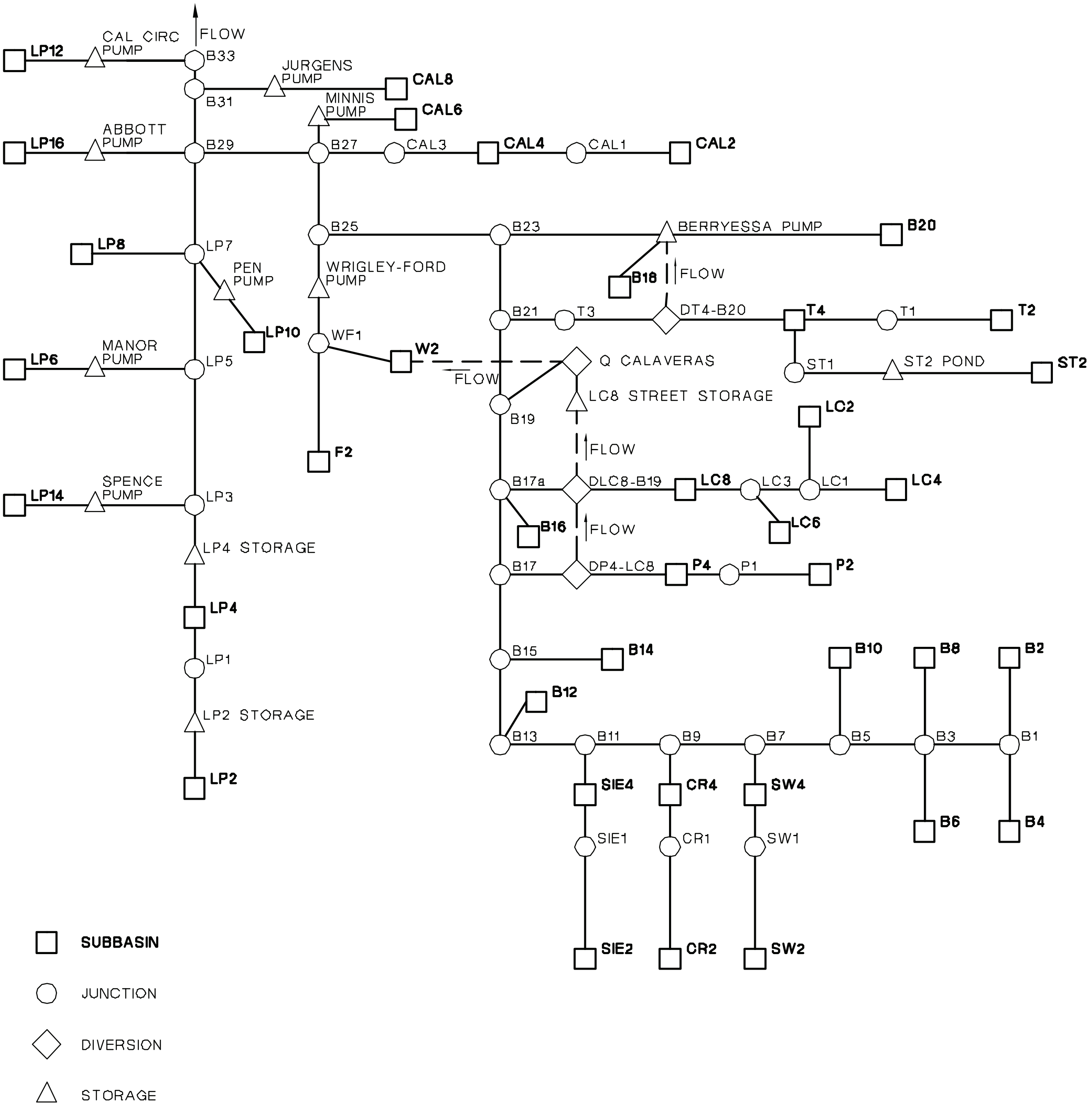
Berryessa Creek Levees Project

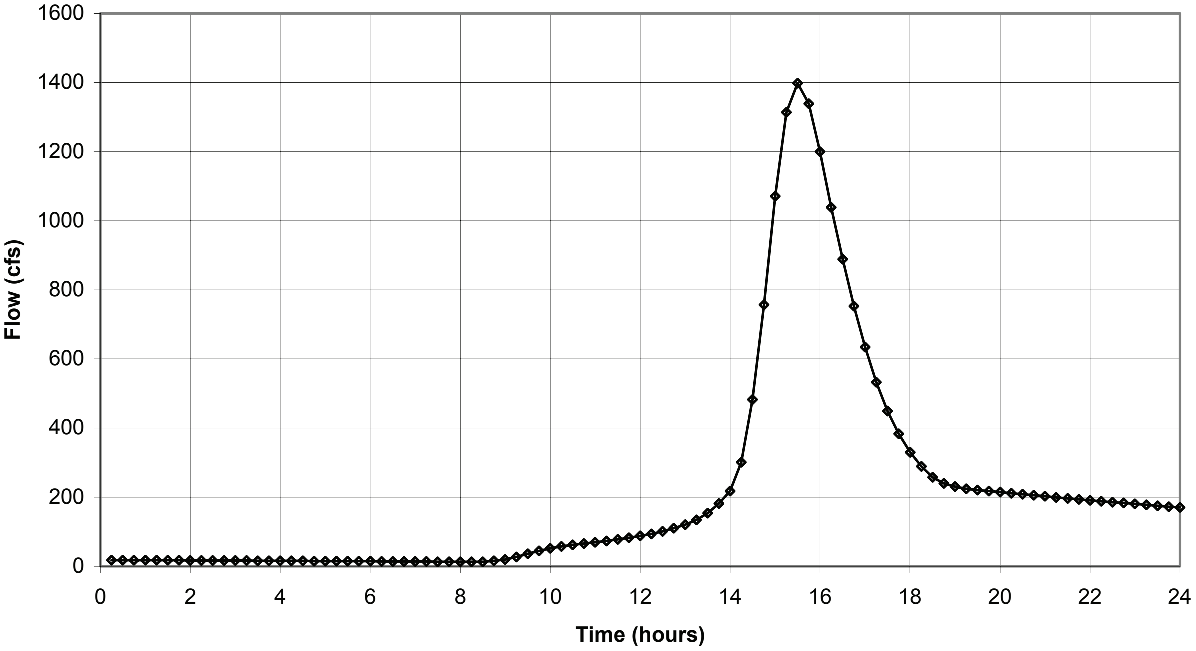
Future Landuse
Figure 6



HEC-HMS Model Schematic
Figure 7

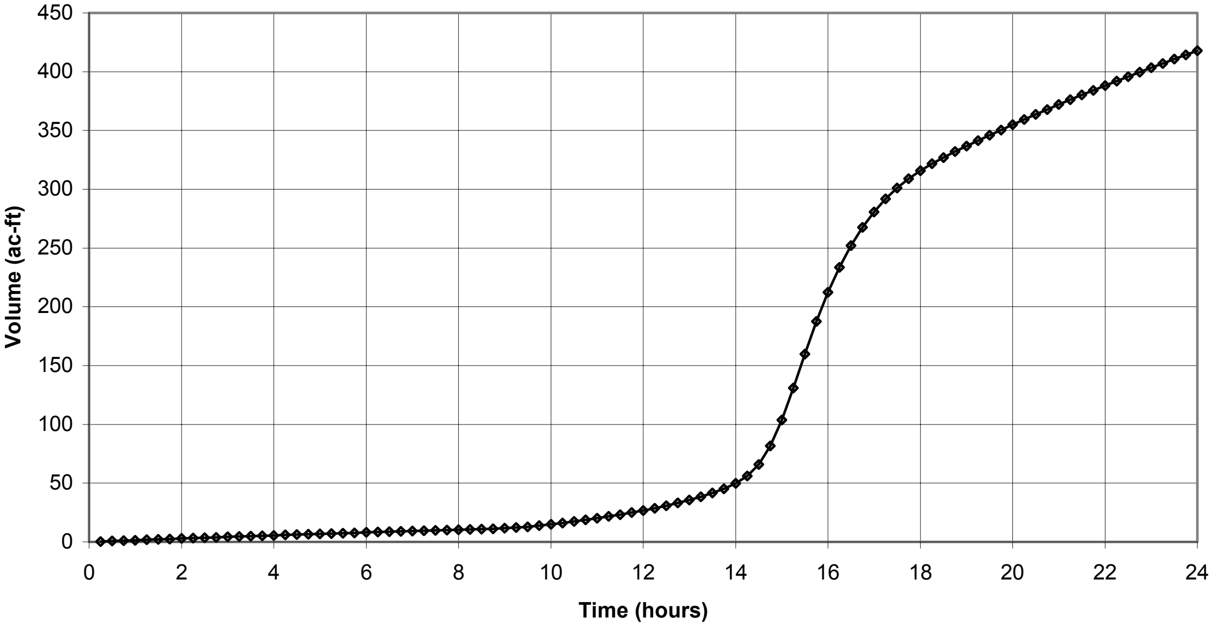
24-Hour, 100-Year Flow

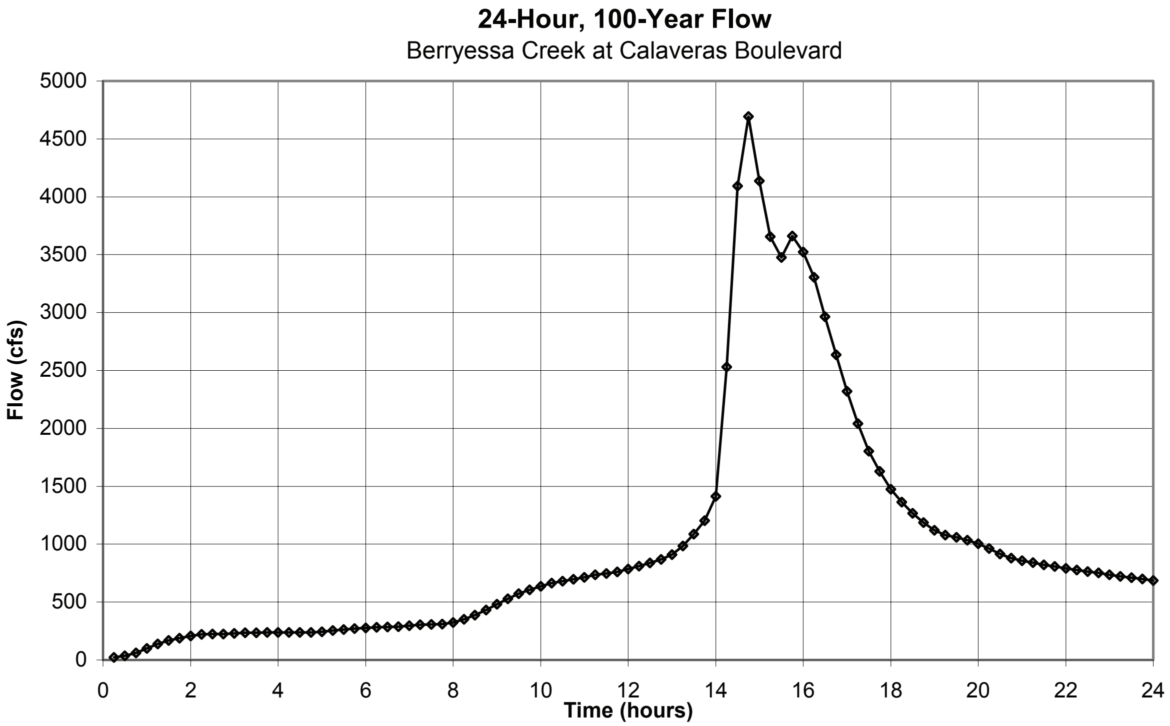
Berryessa Creek at Old Piedmont Road

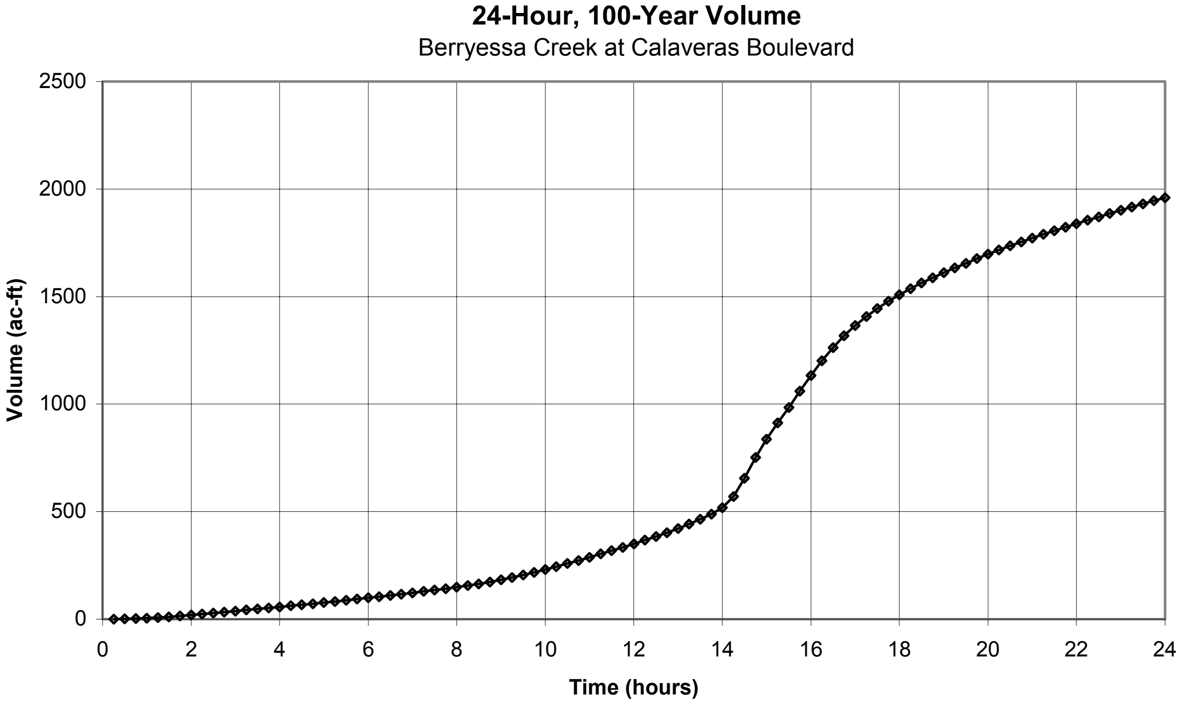


24-Hour, 100-Year Volume

Berryessa Creek at Old Piedmont Road

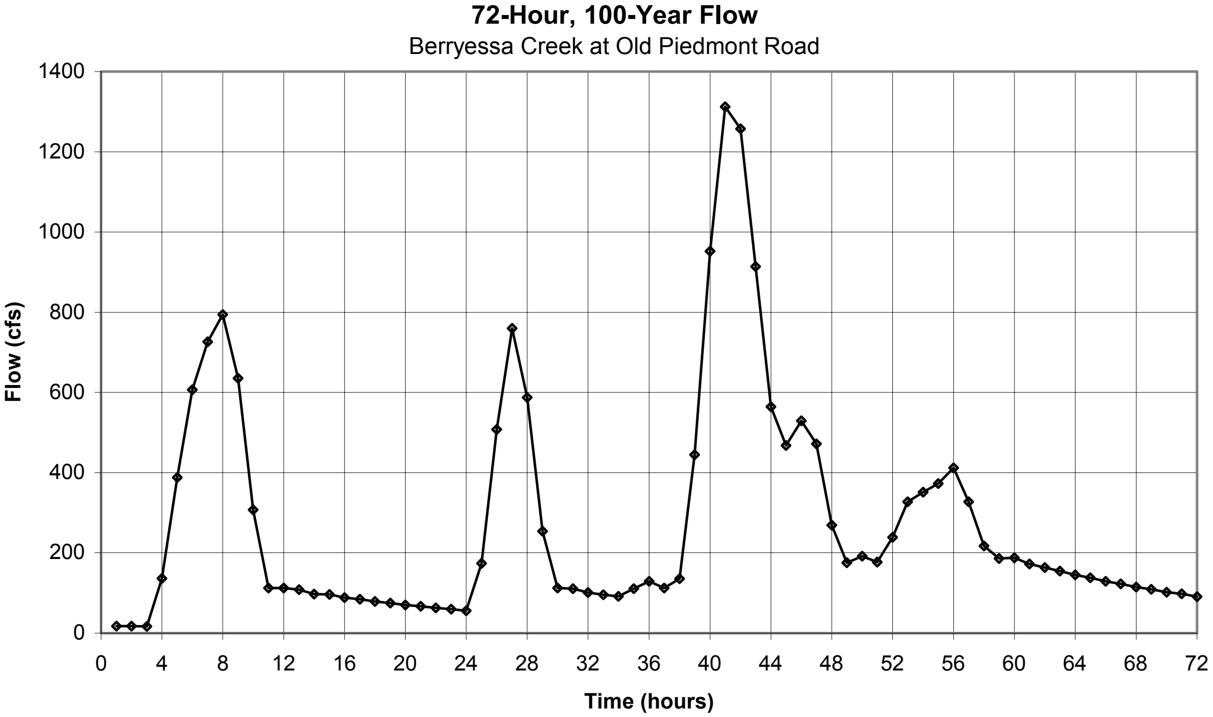




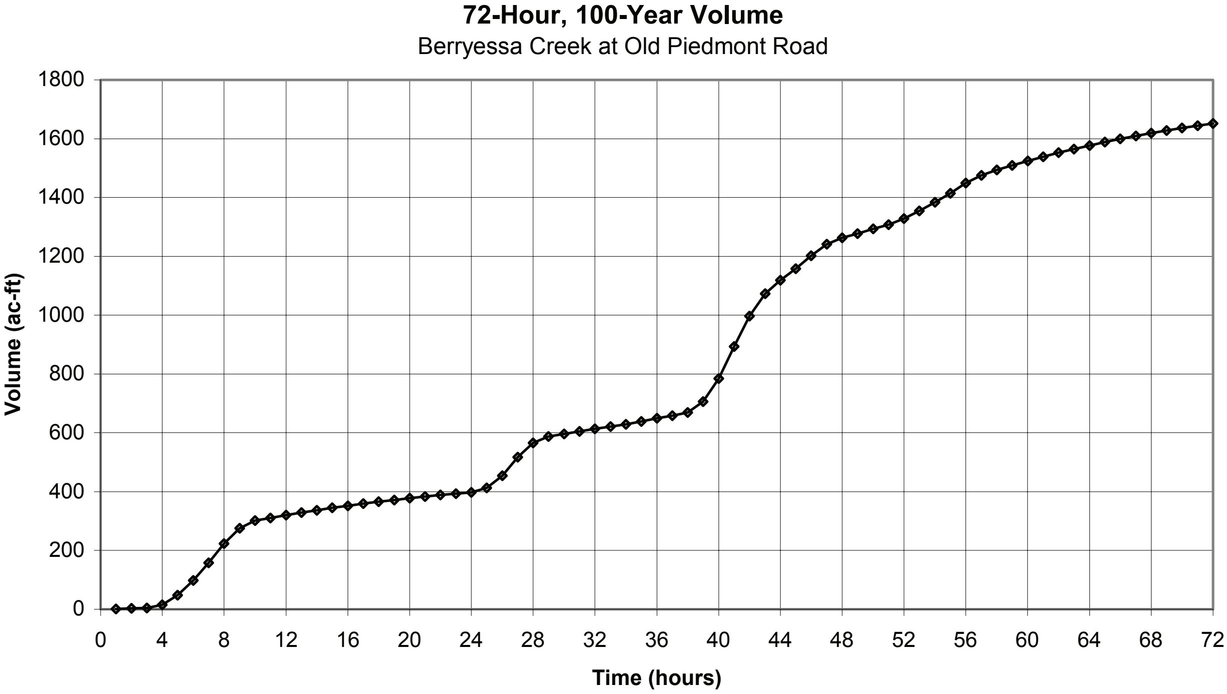


24-Hour, 100-year Model Results at Calaveras Boulevard

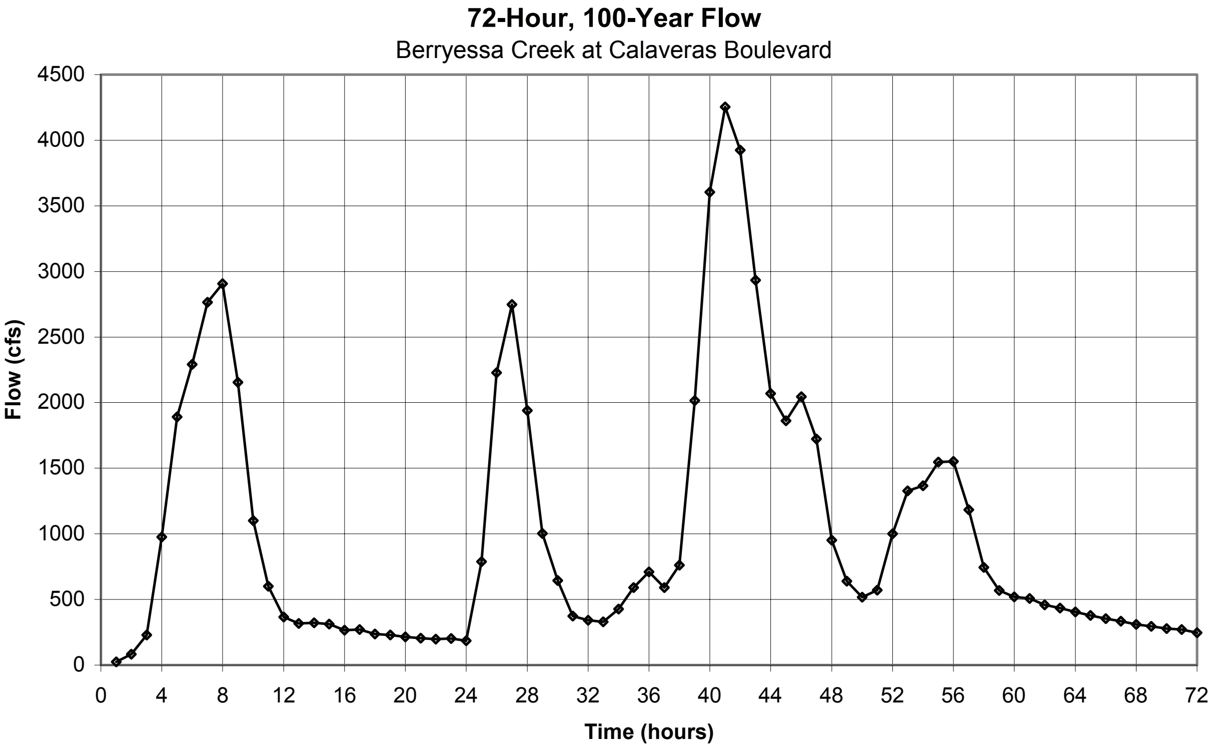
Figure 9

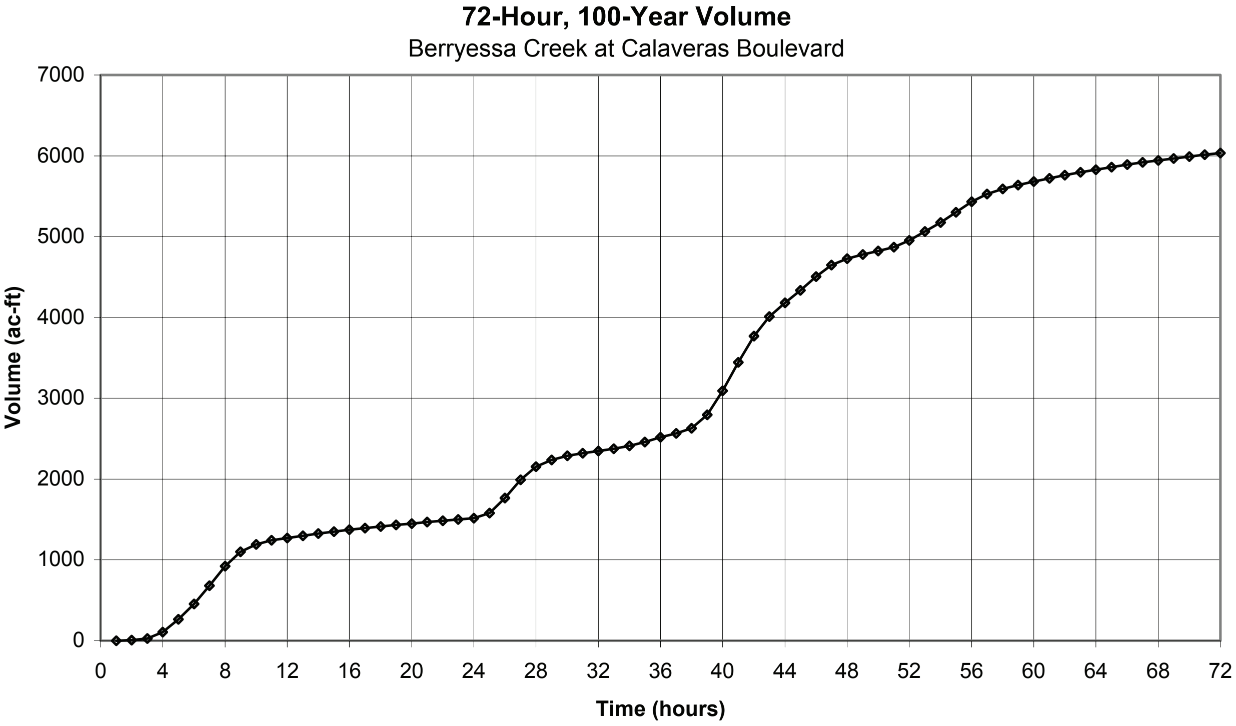


72-Hour, 100-Year Flow
Berryessa Creek at Old Piedmont Road
Flow (cfs)
1400
1200
1000
800
600
400
200
0
0
4
8
12
16
20
24
28
32
36
40
44
48
52
56
60
64
68
72
Time (hours)



72-Hour, 100-Year Volume
Berryessa Creek at Old Piedmont Road
Volume (ac-ft)
1800
1600
1400
1200
1000
800
600
400
200
0
0
4
8
12
16
20
24
28
32
36
40
44
48
52
56
60
64
68
72
Time (hours)


72-Hour, 100-Year Flow
Berryessa Creek at Calaveras Boulevard


72-Hour, 100-Year Volume
Berryessa Creek at Calaveras Boulevard

72-Hour, 100-year Model Results at Calaveras Boulevard
Figure 11